Water Power

NOVEMBER 1957

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The photograph shows the Bersimis River, Quebec.

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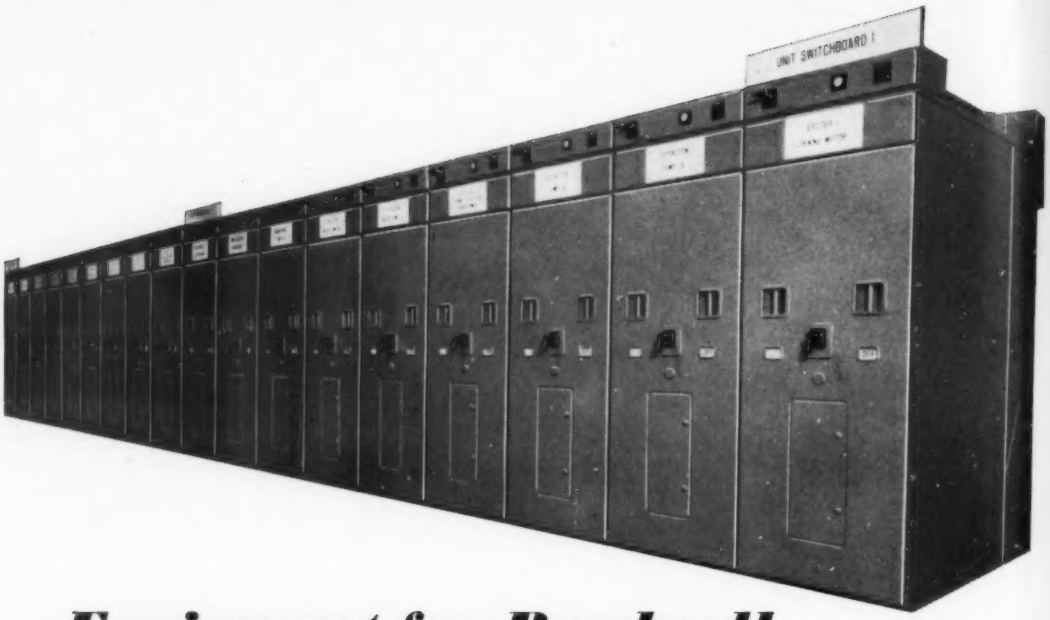

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A technical journal devoted to the study of all aspects of Hydro-electric developments

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Volume 9 Number 11

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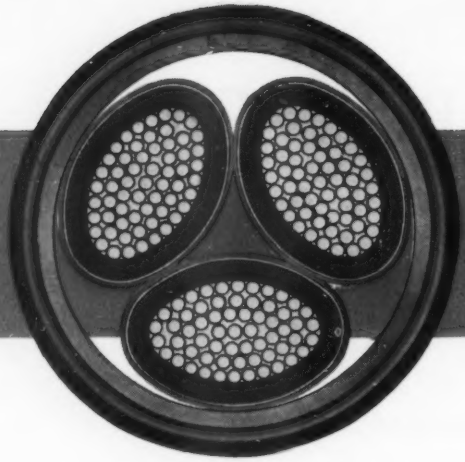

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WATER POWER

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Money from International Sources

IT is significant that an increasing number of engineers are being appointed to the staffs of international financial organisations, such as the World Bank. More and more of the smaller countries are finding that to harness their hydro-electric potentials they must needs approach financial resources outside their own boundaries, since the scale of capital investment to provide for the ever-growing power needs of the future becomes greater every year. In turn, the providers of money need to be assured that their loans are to be wisely spent, and so they insist that their own independent experts are allowed to survey any proposed scheme and to correlate its place in the economy of the country with all other relevant factors.

This trend, which can be seen to be increasing in extent to embrace more countries each year, may well have several important results. First, the breadth of world experience will be brought to bear on the technical and economic feasibility of even the smallest scheme, thus resulting in projects that in general are sounder in design than they might otherwise have been. This is not to suggest that engineers in the smaller countries are not capable of designing their own power projects; but it is inevitable that the wider vision of the international experts can bring advantages not otherwise available.

Secondly, there may well stem from this trend a result which many may think to be of more doubtful advantage. It is likely that he who pays the piper will, as always, call the tune. In other words, the providers of finance will want a considerable say in the directions in which it is spent. This could mean that free competition between the organisations who undertake the construction of dams, and the manufacturers of turbines, generators and electrical plant such as transmission lines, would become nullified. The technical experts, under the tutelage of their governing bodies, might recommend that contracts should be placed in certain directions; and these directions could possibly be dictated, at one or two removes, by political considerations rather than by matters of technical superiority.

Again, those who have the handling of vast sums of money traditionally play safe. They are likely to lean heavily toward those designs of civil-engineering works and those manufactured products which have been well tried out in existing projects; the possibility of experiment, if not ruled out, is likely to become more remote. Standardisation, with its certain economic advantages and its possible technical disadvantages, will almost certainly play a greater part in hydro-electric development. Incidentally, one wonders how international financial experts would have reacted to the French plan to harness the tides at Rance. Here, not only the technique but the very principle itself is new, and the advisers might well have said that they would only consider making a loan, if it had been called for, for a *second* tidal power scheme, provided that the first had proved successful. Fortunately the French Government did not need to consult anyone outside the ranks of its own courageous advisers, and the scheme is going ahead solely on French capital.

So far we have mentioned only hydro-electric schemes, which, on account of their high first cost, naturally need greater capital than thermal power-producing projects. But in many countries the power situation has to be considered as a whole, with thermal and hydro development proceeding side by side. The onset of nuclear power programmes with capital costs per kilowatt of the same order as hydro power may well have the effect of causing even more recourse to the international financing authorities.

Looking ahead, and bearing in mind the thoughts expressed recently by Professor Blackett in his Presidential Address to the British Association in which he said that the gulf between the "have" and the "have-not" countries is rapidly growing, instead of closing, it seems possible that in respect of power production of all kinds and particularly hydro power, the world will be divided into two sections. First there will be the wealthy industrial countries, each able to finance and engineer its own schemes. Then there will be the rest of the world, dominated by one, two or at the

most three great financial influences, very largely dictating the vital power programmes of all the other countries by virtue of the fact that only by their aid can any significant power scheme be built at all.

Such a prospect, which is by no means far-fetched, may be viewed by many engineers with some degree of apprehension: but it is in effect no more than an extension on a world scale of the nationalisation processes that have been apparent for the last three decades in many countries of the world, in regard to power production. In Great Britain, in France, in the U.S.S.R. there already exist completely nationalised and centrally planned power systems. In the U.S.A. the power pools continue to grow so that interconnection—and consequently some degree of nationally co-ordinated planning—is becoming nationwide. In that country, too, the Atomic Energy Commission, from the start of the planning of nuclear stations, has exercised a stronger influence on future power development than ever existed in any form before.

King Canute is supposed to have told the invincible tide to turn back at his command; the more sensible attitude for the modern power engineer is to welcome the tide—and then harness it.

Tully Falls Station Opened

A NEW hydro-electric station at Tully Falls, North Queensland, Australia, has been opened by the State Premier, Mr. Nicklin. The new power station is part of a project which will eventually supply electricity to more than 75,00 square miles of North Queensland. The Tully Falls project is one of the biggest undertaken in Australia since the war, and is designed to provide all power needs in the rapidly expanding north of Queensland for many years to come. The Careeya station, as it is named, is situated underground at the base of the 1,500 ft. Tully waterfall. Power from Careeya will first be fed into the neighbouring areas of Cairns and Townsville, and later extend over a much wider area. The whole project will be completed by 1960, at a cost of more than £A16,000,000.

British Columbia Sanctions World's Largest Hydro Project

THE Government of British Columbia has entered into an agreement with the Wenner Gren Foundation for the development of the Rocky Mountain Trench which forms the arctic drainage of the Peace River. According to The British Thomson-Houston Co. Ltd., technical advisers to the Wenner Gren Foundation, the Trench offers a power potentiality of at least 4,000,000 h.p., so that the scheme will be the largest in the world yet to be undertaken. Indeed, the only comparable scheme is the Hamilton River, Labrador, which has about the same potentiality but about which no decision to develop has yet been made. By damming the Peace River to raise the level in the Trench from 1,970 ft. to about 2,350 ft. a reservoir of about 130 million acre ft. will be created. As regards capacity it will thus share with Kariba—also 130 million acre ft.—the title of the world's largest man-made lake, but the Trench reservoir will be about 260 miles long as against 175 miles in the case of Kariba.

The Peace River development will be of immense value to British Columbia, for it is a hitherto untapped source of power which will add at least 4 million horsepower to the previously estimated hydro potential of 25½ million horsepower, rather over 10% of which has so far been developed. It will also defer the need to develop the Fraser and Columbia Rivers. Fraser River raises the acute problem of interference with the vital salmon industry, whereas the Peace River is devoid of salmon. Furthermore, the development will be entirely under the control of the Province and will not raise international problems as is the case with the Columbia River. The effect on the economic pattern of British Columbia will inevitably be profound; in the words of Mr. R. W. Bonner, Attorney General of British Columbia, "it will roll the habitable and industrial frontier of the Province northward by about 500 miles."

Norwegian Power Production

WHEN the construction of new power stations in Norway now under way is completed in 1961, the production of electric power will be increased by 30%. The Norwegian power stations will then be able to deliver 34 milliard kWh per annum, which is three times as much as in 1945. Before the war 75% of the production of electricity was used by industry. This percentage has now been reduced to 65%—which means that homes, schools, hospitals, etc., now use more of the total output than before. Before the war 21% of the total population had no electricity, but when the new power plants are producing, only 2% will be without electricity. Practically the whole new construction work is being done by the State and Municipalities: 35% by the State and 59% by the Municipalities.

Computer for East Kilbride

THE latest order for a DEUCE computer has been placed with the English Electric Co. Ltd. by the Mechanical Engineering Research Laboratories, East Kilbride. It will be employed mainly in calculations in hydro and thermo-mechanical problems, and will be the first machine of the series to be equipped with high-speed paper-tape as well as punched-card input. This will increase the flexibility of its applications, one of which will be to process paper-tape data from the East Kilbride water tunnel. This will be the fifth DEUCE computer in British Government service.

Ohakuri Diversion Tunnel

EXCAVATION of the 1,350 ft. long diversion tunnel for the Ohakuri hydro-electric station, New Zealand, has been completed by the contractors to the Ministry of Works, the Fletcher Construction Company, and the task of lining it with concrete has begun. This is expected to take several months, and when it is accomplished it will become possible to divert the Waikato River for the construction of the earth dam. In concreting the tunnel the contractors are using the steel shutters, 30 ft. long and 25 ft. in diameter, made for the Maraetai diversion tunnel.

Approximately 500,000 cu. yards of material will be needed for the earth dam which will have a maximum height of 160 ft. with a crest 260 ft. long and

carrying a 22 ft. wide roadway. A forebay channel cut from the rock and varying in width from 130 ft. to 200 ft. by about 500 ft. long will direct the water to a concrete intake dam some 220 ft. long. From this structure the penstocks, each 260 ft. long and 18 ft. in diameter, will lead steeply down to the power house. A separate control building will be sited on the left bank just downstream from the power house.

Using its own forces the Ministry of Works is now well under way with the excavations for the forebay, intake structure, power house and control block. This involves removing about 500,000 cu. yards of rock, but the department hopes to begin the concrete work early next year.

When the new lake is full it will be nearly 5½ sq. miles in area, and will extend for 10 miles upstream. Any surplus water will be discharged well downstream of the earth dam by means of a concrete-lined spillway channel 35 ft. wide, 20 ft. deep and about 750 ft. long.

Ohakuri is the most recently started and the farthest upstream of the chain of power stations now being built on the Waikato by the Ministry of Works for the State Hydro-Electric Department. Provision is being made for an installed capacity of 112 MW, to be produced by four 28 MW machines.

Further Scottish Pumped-Storage Scheme Mooted

THE North of Scotland Hydro-Electric Board has disclosed that tests and site investigations are in progress at two points on Loch Lomond to establish the possibility of constructing a pumped-storage plant that might be of similar size to the Loch Awe project. One site is near the present Loch Sloy power station and the other is at Burn of Mar near Balmaha.

Harrsele Station Commissioned

THE largest power plant to be built by private owners in Sweden in recent years, the Harrsele plant on the Ume River, was put into operation at the beginning of September. It utilises a head of 54½ m. and, when completed, will have three generators with a combined rating of 200 MW. An article on the blasting of the 3,400 m. by 260 sq. m. discharge tunnel was published in our November 1956 issue. The new plant, administered by the Harrsele AB, Stockholm, is owned by a group of private companies with the timber-processing industries Mo och Domsjö, the Swedish Cellulose Company and Nordmalings Ångsåg AB as chief shareholders, together with the South-Swedish Power Company, big private power distributors.

Harrsele is the second station in a series of private power plants in the Ume River which have a joint administration in Stockholm. The first to be completed was Pengfors, downstream of Harrsele, which was built in 1951-55 and in which the third unit was put into operation early this year. It utilises a head of 15·5 m. and has a rating of 51 MW. The station is owned by the South-Swedish Power Company and the Swedish Cellulose Company. Work in the third plant, Bjurfors Nedre, upstream of Harrsele, commenced in July 1956, and the station is scheduled to be in full three-unit operation early in 1960. Utilising a head of 20 m. it will have an installed rating of 72 MW. A planned sister station, Bjurfors Övre, farther upstream, will utilise a head of 10 m. Owners are the Avesta Jernverks AB and the South-Swedish Power Company.

Consulting engineers for constructions and mechanical equipment for all these stations are AB Vattenbyggnadsbyrån, Stockholm, while KMW are supplying the turbines and ASEA the generators and other electrical equipment.

The Ume River will be one of Sweden's most important power-producing rivers. By 1965, the total 350 m. head from the Storuman lake to the outlet into the Gulf of Bothnia will be utilised by no less than 13 power stations with an aggregate installed rating of about 1,250 MW and an annual output of approximately 5,500 million kWh. These stations will include Sweden's biggest power plant, the 375 MW Stornorrfors station, which is at present being built by the Swedish State Power Board.

Bulgaria's Power Resources

THE total installed capacity of Bulgaria's electric power industry was 476 MW in 1956 compared with 106 MW in 1944. Output per capita has risen from 44·6 kWh in 1944 to 76·9 kWh in 1948, 274 kWh in 1955 and 310 kWh in 1956.

Electricity Supply in the Murray Valley

THE N.S.W. ELECTRICITY COMMISSION, Australia, has just linked the new Hume hydro-electric power station near Albury to the main N.S.W. supply system by a 132 kV transmission line, and is commissioning the second 25 MW set at this station, which is already supplying power to the Victorian system. The 72-mile transmission line runs from Wagga to Hume via Albury, where work is well in hand on a major new substation which will provide supply to Albury and district, and subsequently to towns along the Murray Valley.

Completion of the new line will link the supply systems of N.S.W. and Victoria for the first time, though there will not be any major exchanges of power by this means. However, Albury and district, supplied by Victoria for the last 30 years, will be transferred to the N.S.W. supply system when the Albury substation is placed in service, and a number of towns along the Murray Valley will be transferred subsequently. A considerable amount of construction work is involved both in the transfer and in the extension of supply to a number of new areas.

The Albury substation will take supply at 132 kV and step this down to 22 kV for local distribution by the Albury City Council. The first transformer will be delivered to the site before the end of the year and made ready for service, and the substation should be fully operational before next (Australian) winter. Subsequently, the Commission will establish another 132 kV substation at Mulwala, and 66 kV substations at both Finley and Deniliquin. Mulwala, serving as a major bulk supply point for extensions along the Murray Valley, will not be ready for service until 1960, but earlier assistance to the Berrigan Shire area will be provided by a transmission line to be built between Albury and Finley.

Fig. 1. Gioveretto dam and reservoir, having 19,600,000 cu. m. useful storage at 1,850·5 m.

The Plima-Lasa Development

A 55 MVA scheme has just been completed by Società Idroelettrica Atesina to harness two tributaries of the Adige in Northern Italy

THE Plima-Lasa scheme, which has just been completed by Società Idroelettrica Atesina, an associate company of the Montecatini Group, forms part of the development of the upper Adige basin above Merano. The Plima, which flows through the Val Martello, and the Lasa are right-bank tributaries of the Adige and descend from the northern flank of the Cevedale massif.

The development is a high-altitude one, for most of the works lie between 1,800 and 2,000 m., and the total lack of communications with the valley of the Lasa presented serious obstacles to the execution of the works.

A 55 MVA station at Lasa utilises about 90 million cu. m. of water per annum at a head of about 983 m. from a reservoir formed in the Martello valley near Gioveretto by a buttress dam 83 m. high, the maximum retention level being at an elevation of 1,850·5 m. The discharge from the station passes immediately into the supply tunnel for the Castelbello station owned by Società Montecatini.

A map of the catchment area, showing the plan of development, is given in Fig. 2. A total catchment area is utilised of 117·4 sq. km. at an average elevation of 2,400 m. A mean annual flow is available of 2·41 cu. m. per sec. and a maximum annual flow of 7 cu. m. per sec.

The Gioveretto reservoir has a live capacity of 19,600,000 cu. m. and feeds a supply tunnel extending 6,000 m. to the Lasa intake, from which it is carried a further 5,000 m. to a surge chamber whence a steel penstock 2,180 m. long leads to a surface-type station housing a single horizontal generating set of the twin Pelton-wheel type. Five downstream tributaries of the Plima are diverted to the reservoir.

The scheme is capable of a mean annual production of 200 million kWh.

Gioveretto Dam

Various views and drawings of the Gioveretto dam are given in Figs. 1, 3, 4 and 5. It is of the buttress type comprising 17 buttresses between a gravity section at either end. It is 380 m. long at the crest, 83 m. above lowest foundation, contains 310,000 cu. m. of concrete, and entailed 250,000 cu. m. of excavation. A spillway in the left-hand gravity section and a bottom-discharge tunnel in the left bank combine to give a total flood-discharge capacity of 500 cu. m. per sec. As a matter of interest, 65 cu. m. of water are impounded per cu. m. of concrete, representing 200 kWh.

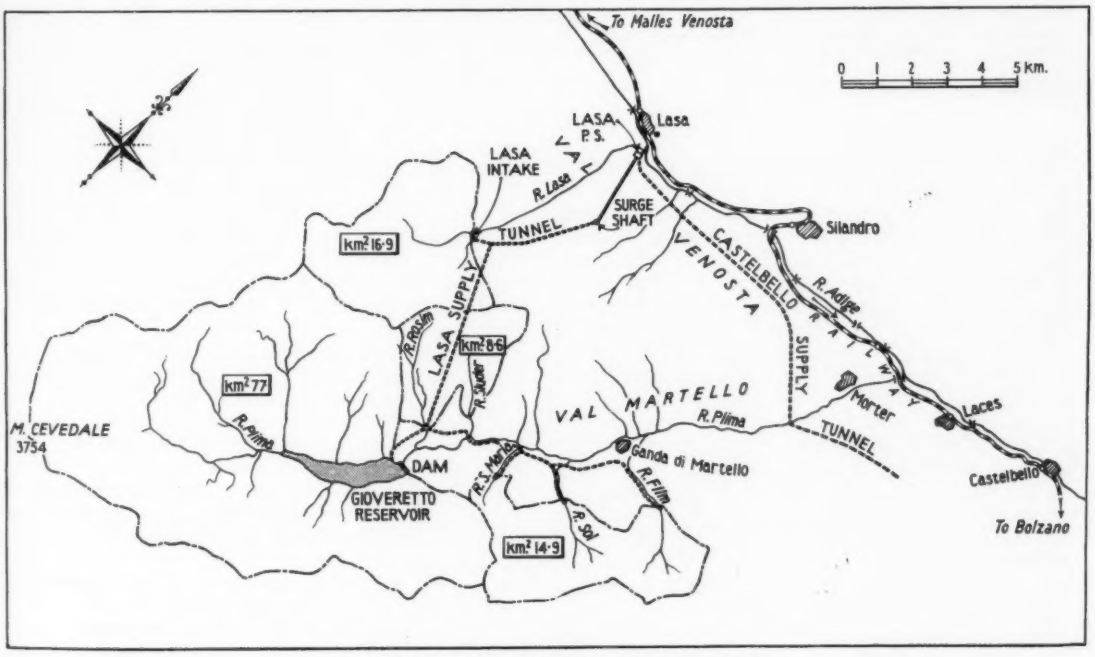

Fig. 2. Map of the Plima-Lasa Development

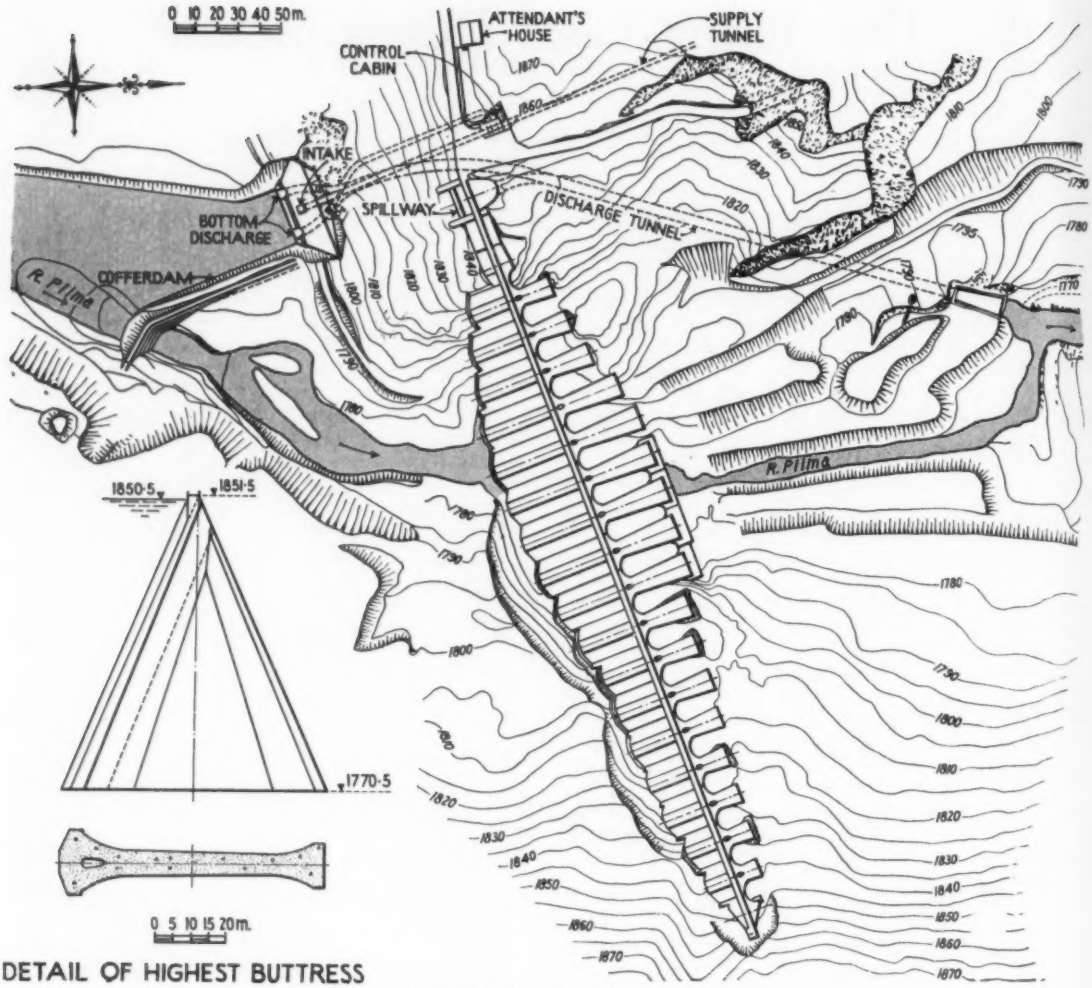

Fig. 3. Plan and detail of Gioveretto dam

The dam is founded on a glacial till, and entailed careful geological investigations and meticulous design studies. The resulting design contains features that are thought to be unique. As will be noted from Fig. 3, the buttress head contains a cavity, which is of great benefit in dissipating the heat of hydration, reduces the area subjected to uplift pressure, and economises in concrete. As compared with a simple gravity dam, the saving in concrete ranges from 29% for the 50 m. buttress to 36% for the 80 m. buttress. The composition of the concrete has been chosen to achieve maximum impermeability and resistance to ice damage, combined with good workability. This has made it possible to dispense with the stone facing on the water side of the dam usually necessary at these altitudes. The curved surfaces of the buttresses are based on circular arcs, permitting the use of simple shuttering and favouring an economical layout. Another departure in design is that the sealing joints between buttresses are made of rubber.

Once the type of structure had been chosen, the Civil Engineering Branch of Montecatini's Power Department, under the direction of Dr. Ing. Dante Finzi, successively studied no less than 16 solutions before the definitive one. Although the basic assumptions involved in the stability analysis are the same as those adopted for simple gravity dams, the computation of the principal stresses was very laborious, and to evaluate these stresses Dr. Finzi's assistants, Dr. Ing. Antonio Ghirardini and Dr. Ing. Carlo Niccolai, developed an original analytical procedure.*

When the final design had been selected, the behaviour of the 80 m. buttress was checked by model tests under the direction of Professor G. Oberti at the ISMES Laboratory in Bergamo.

In the construction of the dam, use was made for the first time in Italy of elutriation of the sand to eliminate dust below a certain size and thus remove a source of uncertainty in the final strength of the concrete. The system adopted was the RHEAX, already described in this journal.†

* See "Contributo al Calcolo delle Dighe a Gravità Alleggerite." By Dott. Ing. Antonio Ghirardini and Dott. Ing. Carlo Niccolai. *L'Energia Elettrica*, Vol. XXXIII, No. 9, 1956.

† See "Criticism of Elutriation Systems for the Treatment of Sand for Concrete." By Dr. Th. Eder. WATER POWER, June 1954, p. 217; and "New Experience in Mass Concrete Construction." By Dipl. Ing. Dr. Josef Fritsch. WATER POWER, July 1954, p. 258, and August 1954, p. 304.

To enable the actual behaviour of the dam to be compared with the results of the computations and with the model tests, 336 electrical gauges have been embedded in the 80 m. buttress, comprising 135 strain gauges, 184 thermometers, and 14 humidity meters.

The bottom-discharge tunnel in the left bank has two intakes, one on either side of the supply-tunnel intake structure, each controlled by a 3·4 by 2·8 m. sluice. The surface spillway is controlled by a flap gate, 15 m. wide by 4·5 m. high, and the discharge passes down a sloping tunnel, 85 m. long, to join the bottom-discharge tunnel.

Supply Tunnel and Penstock

The supply-tunnel intake works are placed at an invert level of 1,792·35 m. A tunnel 2·5 m. in diameter and 92 m. long, the mouth of which is covered with a fixed sloping grid, extends to the control shaft, where the water passes through a basket-type screen followed by a flat screen, after which entry to the tunnel proper is controlled by a sluice 1·5 m. wide and 1·8 m. high at the throat of a venturi section.

The supply tunnel is 2·1 m. in diameter and is subjected to a maximum hydrostatic pressure of 70 m. It is lined with concrete throughout, and in certain sections the concrete is reinforced.

The waters diverted from the downstream tributaries of the Plima are conveyed in a buried steel pipeline along the river valley and flow into the supply tunnel near the intake from the first of these tributaries—the Rosim. This pipeline is considerably below the level of the supply tunnel and operates as an inverted syphon.

Drawings of the intake shaft from the Lasa, which enters 6,000 m. down the supply tunnel, are given in Fig. 6. The Lasa waters have to descend about 73 m. to reach the tunnel, and their flow is restrained by a series of baffles built into a sloping shaft. This arrangement, which has been employed in several Montecatini schemes, is claimed to eliminate the danger of air entrainment in the tunnel.

The arrangement of the surge chamber is shown in Fig. 7. It is of the two-chamber restricted-orifice type, and is succeeded by

Fig. 4. Gioveretto dam under construction, showing the hollow-buttress arrangement

Fig. 5. Detail of the climbing shuttering

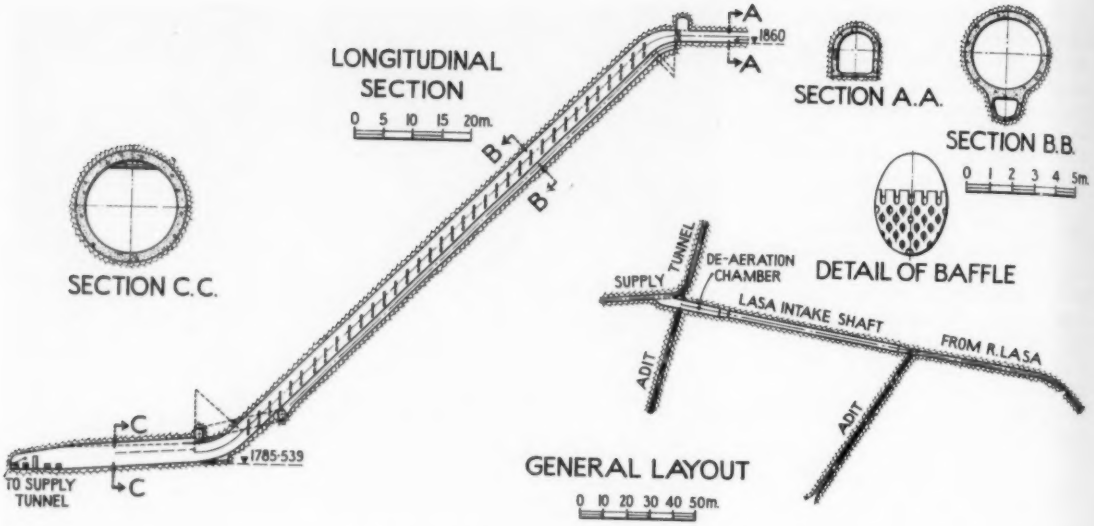

Fig. 6. General arrangement of the Lasa intake

a valve house containing two 2 m. diameter butterfly valves, one of which is for normal use, whereas the other is for emergency.

The penstock is 2,180 m. long and ranges in diameter from 1·70 to 1·47 m. Over-pressured steel pipes are used, in lengths of 6 to 8 m. and of a maximum individual weight of 8 tons, the total weight of the penstock being 2,150 tons. An aerial ropeway extends over the whole length of the track, and the penstock is buried in a trench.

Lasa Power Station

Interior and exterior views of Lasa power station can be seen in Figs. 8 and 9. It is a simple but modern design, 45 m. long, 16 m. wide and 16·5 m. high.

It contains a single generating set consisting of a horizontal alternator at either end of which is an overhung single-jet Pelton wheel. The turbine is designed for an output of 53,200 kW at 500 r.p.m. at a net head of 951·1 m. and a flow of 6·5 cu. m. per sec. Each wheel is supplied by a Riva straight-flow nozzle preceded by a Riva spherical shut-off valve.

The alternator is a 12 pole Siemens-Schuckert closed-circulation machine rated at 55 MVA, 0·8 p.f., 10 kV, with a flywheel effect of 610 ton-m². It is furnished with differential protection and with protection against overcurrent, overvoltage, overload, and stator and rotor earthing. A rapid-action voltage regulator can control the voltage between 8 and 11 kV, and embodies reverse excitation to control the charging current when energising a long high-voltage line.

Aluminium busbars convey the current from the generator through a gallery to a transformer in the adjacent switchyard, a 15 kV 1,150 MVA compressed-air circuit breaker being interposed between generator and transformer. The transformer is of the three-winding type, stepping up from 10 kV to 130 and 220 kV. (The actual transformer ratio, referred to no-load, is 10/135·1/240·9 kV, at a rating of 57·6/57·1/57·6 MVA.)

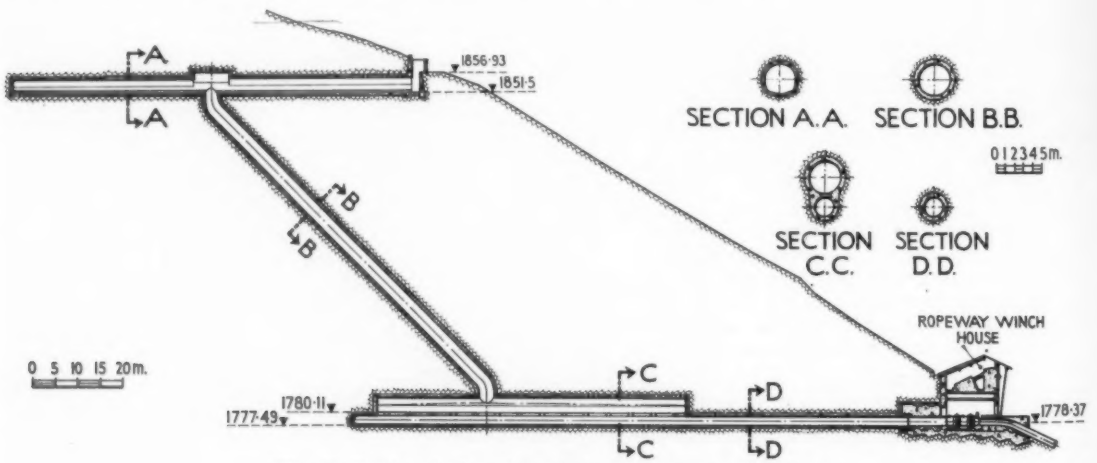

Fig. 7. Sectional elevation of the Lasa surge chamber

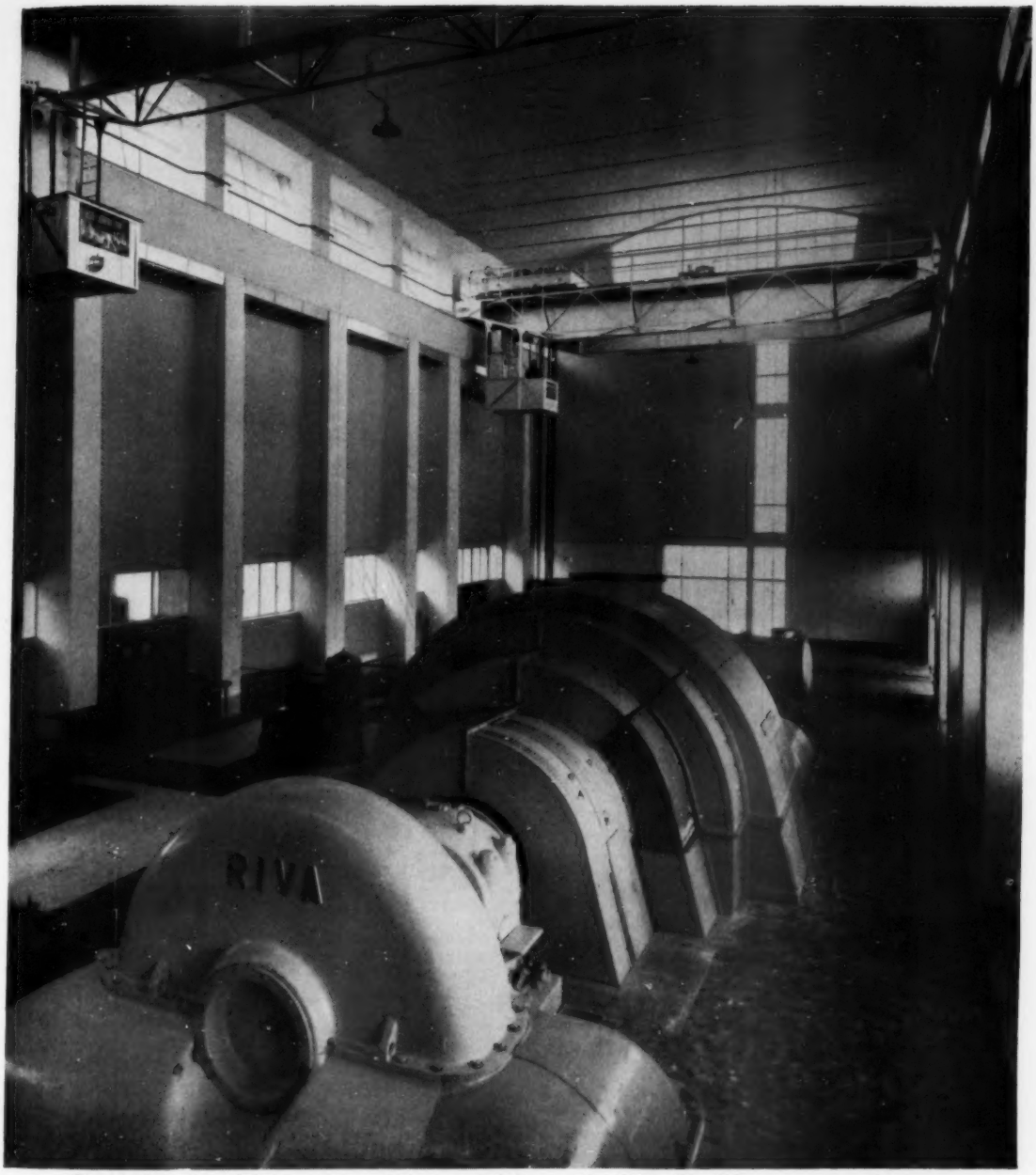

Fig. 8. Interior of Lasa power station containing a 55 MVA Riva/Siemens generating set

The circuit breakers on the high-voltage side are intended merely to control their respective transmission lines; there is one line at each voltage and provision is made for a further line. These circuit breakers are of the minimum-oil type and are arranged for high-speed automatic reclosing. The 220 kV breaker is rated at 4,000 MVA, and the 130 kV breaker at 3,000 MVA.

Auxiliary power is derived from the generator busbars and stepped down to 220 V in a 250 kVA transformer. A second similar transformer connects the auxiliary supply with an independent 10 kV busbar which can receive power from Castelbello and Glorenza and supplies feeders to Lasa and to the intake works.

All control is centred in a control room in an annexe at one end of the station, commanding a view both of the machine hall and of the switchyard.

Acknowledgments

The Plima-Lasa development was designed and constructed by Montecatini Power Department, and this article is published with the kind permission of Società Idroelettrica Atesina, the associate company

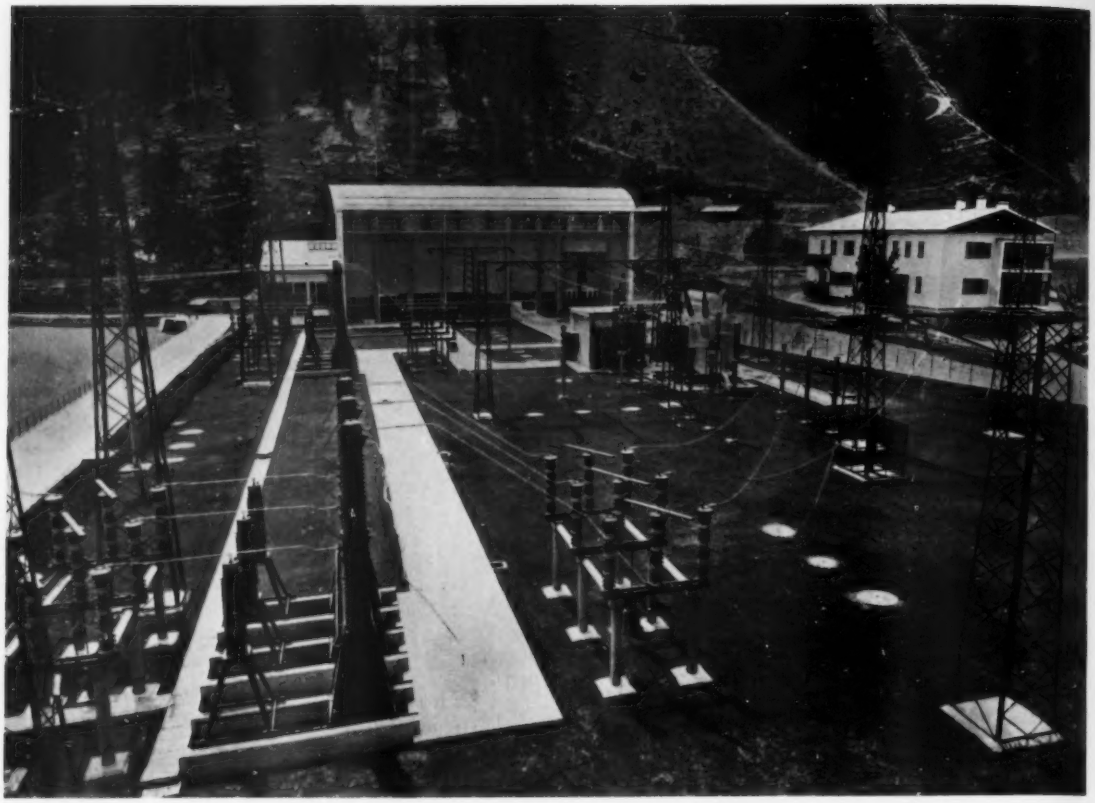

Fig. 9. A view of Lasa station and switchyard

owning the plant. We should also like to express our indebtedness to Electroconsult, Milan, the consulting organisation established by Montecatini, Fiat and SADE, who have placed all the necessary technical information at our disposal. The following is a list of the contractors and manufacturers concerned:—

CONTRACTORS

Dam: Excavation and Concreting
Società Idroelettrica Atesina, Bolzano
Dott. Ing. Giuseppe Torno & C., Milan
Dam: Drilling and Grouting
Impresa Costruzione Opere Specializzate (ICOS), Milan
Tunnel: Excavation and Concreting
Società Idroelettrica Atesina, Bolzano
Tunnel: Grouting
La Geotecnica, Milan
Penstock and Power Station
Società Idroelettrica Atesina, Bolzano

MANUFACTURERS

Gates and Sluices
S. A. Alessandro Calzoni, Bologna
Soc. Naz. Officine di Savigliano, Turin
Penstock
Cofor-Terni, Terni
Turbine
Costruzioni Meccaniche Riva S.p.A., Milan
Alternator
Siemens-Schuckertwerke, Erlangen
Transformers
Compagnia Generale di Elettricità, Milan
Crane
Brighi, Carnovali & Ronchi, Milan
Control Room and Protection Equipment
C.G.S., Milan
Tecnomasio Italiano Brown Boveri, Milan
Compagnia Elettrotechnica Italiana, Milan
Switchyard: Structure
Società Anonima Elettrificazione S.p.A., Milan
Circuit Breakers
Magrini S.p.A., Bergamo
Bagnagatti, Milan
Officine Elettromeccaniche Galileo di Battaglia Terme S.p.A., Battaglia Terme (Padua)
Scarpa & Magnano, Milan
Measuring Equipment for the Dam
Ph. Instr. Huggenberger, Zurich
Officine Galileo di Milano, Milan

Symposium on Underground Stations. In connection with our summary of papers, listed on page 377 of our October issue, read at the recent Symposium on Underground Stations organised by the American Society of Civil Engineers, we are now informed that Dr. Ing. Mario Mainardis, M.A.I.E.E., Chief Engineer, Electromechanical and Operation Department, S.A.D.E., Venice, was a co-author with Dr. Ing. Carlo Semenza and Dr. Ing. Dante Finzi of the paper "Underground Power Plants in Italy, with special reference to S.A.D.E. and Montecatini Power Stations."

Switchboard Instruments. A series of leaflets from The English Electric Co. Ltd. relates to switchboard instruments of round, square, projecting and flush types.

Developments on the Dnieper

Brief particulars of the new power prospects envisaged by the latest Five-Year Plan

IN the February (1957) issue of *Gidrotekhn. Stroit.* (Hydraulic Construction), E. A. Baksheev describes existing and prospective water power development on the river Dnieper. This river is about 1,370 miles long (2,285 km.), with a fall of 220 metres, a run-off area of 503,400 sq. km. and a mean annual flow of 53,000 million cu. m. It rises in the Smolensk district, flows through the White Russian and Ukraine republics, and just south of Kherson enters the Black Sea. The first and still by far the largest hydro-electric power station is that of Dneprovsk (Lenin) with an annual output of 3,612 million kWh. During the past few years further development has been seriously studied by the Ukrainian branch of the Inst. for Hydro-power Planning in conjunction with other bodies.

The flow of the river is very largely affected by snow, and the flow during the spring thaw represents some 60 to 70% (even 80%) of the annual total, with a corresponding sharp drop in mid-summer. Thus, just below Kiev the flow varies from 23,500 cu. m. per sec. in spring to 174 cu. m. per sec. in summer. Some flow figures and power potentials at different points are given in Table I.

Power developments include 14 possible power stations:— Dorogobuzhsk, Smolensk, Orshinsk, Mogilevsk, Vilyakhovsk, Zhlobinsk, Rechitsk, Lyubechsk, Kievsk, Kanevsk, Kremenchugsk, Dneprodzerzhinsk, Dnepro, Kakhovsk. Many of these are named after near-by towns, but only in two cases is the probable capacity mentioned—Kremenchugsk 450,000 kW., and Kakhovsk 312,000 kW.

The relatively copious storage reserves of the Smolensk station—the useful volume of which constitutes about 24% of the mean annual flow of the Dnieper—ensure control for all the proposed downstream stations as far as Sozh (Soj). Below this point, control of the flow will be governed by the storage capacity of the Lyubechsk station. All the other hydro-electric stations of the Upper Dnieper are assumed to have reservoirs which are too small to involve appreciable storage. In the series of hydro-electric stations on the Lower Dnieper, flow control is ensured by the ample storage capacity of the Kremenchugsk plant. Flow control of waters in the river Pripyat basin (one of the Dnieper's main tributaries) will be effected by constructing a series of reservoirs with a total capacity of about 3,000 million cu. m., together with a large number of smaller lakes having a total capacity of 1,700 million cu. m. In connection with these it is intended to build about 80 small hydro-electric stations totalling some 60,000 kW, with a mean annual output of 270 million kWh. Other related works include deepening river beds or other necessary measures for improving river transport and for general agro-technical purposes.

Control of the waters of the river Pripyat will effect conditions to some extent on the Dnieper below Kiev and therefore the operation of the Lower-Dneprovsk station. It has been calculated that this will increase the mean annual output of the Kiev and Kanevsk installations by something like 100 million kWh. But at the Kremenchugsk and other stations of the Lower Dnieper the effect will be relatively small.

The Dneprovsk (Lenin) station was built in 1932 and was the first of the Dneprovsk series. After the serious damage inflicted during the war it was rebuilt with an enlarged storage capacity and the generating plant was increased to 650,000 kW.

In the fifth 5-year period, 1951-55, construction of the Kakhov plant was completed, the reservoir being extended until it backed as far upstream as the Dneprovsk station. In 1954, with a view to further extension of the southern power system and improving the operation of the Dneproges, the construction of the Kremenchugsk station was begun, with sufficient storage to effect major control of the river flow. It was thus possible to raise the output of Dneproges and Kakhovges by some 700 million kWh per annum. What is called the "guaranteed" capacity of the Dneproges was increased from 150,000 to 250,000 kW.

In accordance with the sixth 5-year plan, 1956-60 work was commenced on the Dneprodzerzhinsk

TABLE I

Feature	At junction of tributary				Mouth of river
	Smolensk	Druts	Pripyat	Desna (Kiev)	
Distance from mouth (km.)	1,873	1,444	1,064	968	0
Mean annual flow (million cu. m.) ...	3,000	6,300	32,000	43,000	53,000
Percentage total flow	5·7	11·9	60·5	81·2	100
Potential power reserves (a) MW	10·4	57·1	164·1	246·0	1,652
(b) Million kWh	91	499	1,436	2,154	14,471

TABLE II

Name of station	Working capacity of reservoir, milliard m³	Total area flooding 1,000 ha	Mean annual million kWh	Capital charge per kWh, roubles
Kiev	1·9	142	682	2·72
Kanev	2·0	89	783	1·81
Kremenchug	9·0	224	1,475	1·78
Dnepro-dzerzhinsk	—	48	1,245	1·48
„ ges (Lenin)	—	—	3,612	—
Kakhov	6·8	221	1,449	1·94

station, including also that of the smaller Kanevsk plant. With the completion of these two stations it is reckoned that utilisation of the potential powers of the river Dnieper will be increased by about 85%.

The Kiev and Lyubech (Lubeck) stations would markedly affect the measures to be adopted for reclaiming or draining the Polesia region; so that the question of dates for their construction must be decided in conjunction with these drainage schemes, as part of the general improvement and development of Polesia (on the south side of the middle course of the Pripyat—Pripet). Some particulars of the series of power stations on the Lower Dnieper are given in Table II.

Correspondence

Tunnelling Records

To the Editor of WATER POWER,

In your October issue Mr. G. C. Cummings challenges a contractor's claim for establishing a world record in tunnel driving. I am fairly certain that none of the very fine achievements mentioned by Mr. Cummings are *world* records; a great many factors have to be taken into account to prove that a record has been established. It would be an interesting, though a difficult job to list the so called "records" claimed during the past few years, taking into account the size of tunnel, the number of men employed, rock conditions and many other factors.

In 1955 the most impressive scale of rock tunnelling was to be found in Sweden and details of the Harrsele project have been given in both the editorial and the advertisement columns of your journal. Mr. Cummings claims the 557 ft. driven by his firm in 7 days to be a world record; Swedish contractors claim their 585 in the month of August 1955 to be the world beater. I am inclined to think the latter is a world beater. During this year I have been connected with a tunnel of 9 ft. 6 in. bed width which averaged 247 ft. per week for 65 weeks and had a maximum 7 day drive of 373 ft. This was also a very fine achievement in rock which necessitated using 5·58 lb. of explosive per cubic yard excavated; seventy (70 No.) rounds were fired between 8.00 p.m. Sunday to 8.00 p.m. the following Sunday. This was a determined although not a planned effort and I think, all things considered, constituted another "record" for 1957.

In my opinion rock-tunnelling records should be based on the average footage driven per week throughout the whole job; furthermore it is fast becoming a waste of effort to put forward a claim for a world record as with modern techniques and equipment, "records" are being equalled or broken daily. I suppose for publicity value a well-planned effort for seven days, with all manpower keyed up, is of some use to the contractor, but I cannot believe that after the sustained effort the progress per week would reach even 50% of the "record" figure.

L. E. TAYLOR,
Resident Engineer, Orrin Tunnel.

Russian Engineers Visit Britain

Following the visit last year of a team of British hydro-electric experts to Russia, a team of nine Russian engineers headed by Mr. F. G. Loginov, Chief Deputy of the Ministry of Electric Power Stations of the U.S.S.R., has just completed a return visit to power stations and engineering works in Britain. Nine of the North of Scotland Hydro-Electric Board's stations were visited, including Loch Sloy and stations of the Conon, Garry, Moriston and Breadalbane schemes. Visits were also paid to Calder Hall atomic power station, Drakelow steam station, the laboratories at East Kilbride, Wallingford and Leatherhead, and to the works of John Brown & Co. Ltd., Metropolitan-Vickers Electrical Co. Ltd., and The English Electric Co. Ltd. The visitors expressed their high opinion of the standard of design evidenced at the Scottish stations, and particularly of the means adopted to secure low-cost construction and of the thoroughness with which the fish problem had been tackled. They spoke well of the quality of manufacture of transformers, waterwheel generators, and water turbines. In connection with the British world record of 130 MW for waterwheel alternators they claimed that machines of similar or larger capacity had already been developed in Russia. It appears, however, that the largest machines so far actually constructed are the 105 MW sets for Kuibyshev, although sets of 200 MW are planned for Braeski.

New Czechoslovak Standards for Concrete

A digest of the requirements of the new Czechoslovak standards for concrete to be used in hydraulic structures is given by Dr. Ing. Juraj Stork, Deputy Director of the Institute of Building and Architecture of the Slovak Academy of Sciences, Bratislava

PART TWO

ČSN 73 20 22—Sampling and Scope of Tests

Standard ČSN 73 20 22 includes a specification of the procedure of cement sampling from railway shipments from bins and stores. The weight of a cement sample is 6 kg. for a partial test and 18–20 kg. for a complete test. The procedure of sampling is further established for aggregates transported by conveyors or chutes, by railway cars, from stockyards, and from open or borrow pits. The size of the sample is governed by Table VII. The sampling of water is specified for run-off water, pumped water or well water. Similarly the sampling of earths from the inundation area is specified.

The sampling of concrete mix is specified for mixes transported by conveyors or chutes, from mixers, transport facilities and from the placing site. Hardened concrete is sampled by core-drilling, cutting of the samples by quarry saws or by chiselling. The concrete to be sampled must be at least 28 days or 60 days of age if Portland cement or slow-hardening cement respectively are used.

Scope of tests. For each 1,000 g. of cement the following tests have to be made: initial and final set, volume stability, fineness of grinding and abbreviated test of standard strength. In addition, complete tests of standard strength are to be made for each 2,000 g. and tests of hydration heat, chemical and compound analysis, autoclave expansion test for each 10,000 g. of cement delivered.

Mixing water, if clear, is tested in advance for content of organic impurities, acidity, basicity and sulphate content. If any of the tests are unsatisfactory, the samples have to be chemically analysed. These tests have to be made immediately whenever the source of water or its character is changed.

Aggregate is tested for contents of clay and organic impurities for each 600 cu. m. of aggregate, for moisture content and grading at least for each 300 cu. m. Specific and unit weight is determined for each 5,000 cu. m., and water-absorbing and sulphate content is to be determined for each 5,000 cu. m. Whenever structures including more than 10,000 cu. m. of concrete are concerned, it is necessary to determine the content of clay and clay lumps accurately for each 2,000 cu. m. Content and nature of lightweight aggregate (specific gravity less than 2·0 g./cm^3.) is to be determined for each 5,000 cu. m. Mica content is to be determined for each 5,000 cu. m., and durability for each 20,000 cu. m.

For structures including more than 100,000 cu. m. of concrete tests have to be made for the percentage of grains having suitable shape for each 5,000 cu. m., and percentage of amorphous SiO_2 for each 20,000 cu. m.

The concrete mix is to be tested for consistency for each 100 cu. m., for unit weight and compactibility for each 300 cu. m., for tendency to segregation for each 1,000 cu. m., and for air content (if air-entrained concrete is used) for each 100 cu. m. Hardened concrete is to be tested according to Table XIX.

ČSN 73 20 23—Testing of Concrete Components

Cement tests. Cement tests are specified by general cement standard ČSN 1213. The present standard specifies only autoclave test in agreement with corresponding B.S., alkali-aggregate reaction test and test

TABLE XV.—OVERLAPPING OF FRACTIONS

Fraction $d_{min.}$ to $D_{max.}$	Screen size in mm.					
	Screen next to $D_{max.}$	$D_{max.}$	0·5 $D_{max.}$	$d_{min.}$	0·2	0·1
mm.	Oversize or undersize by weight, %					
0/5	100	90 to 100	—	—	—	—
5/10 to 30/60	100	95 to 100	40 to 65	10	<3	—
30/70 to 80/150	100	95 to 100	40 to 65	15	<3	<1

TABLE XVI.—PERMITTED QUANTITY OF DELETERIOUS SUBSTANCES IN FINE AGGREGATE

Deleterious substances	For durable concrete for hydraulic structures, ČSN 73 20 20	For other concretes for hydraulic structures
Clay, loam and dust particles of aggregate, determined by sedimentation (by weight), including caolinitic clay, montmorilonitic clay	max. 3·5 % max. 1·0 % max. 0·2 %	max. 5·5 % max. 2·0 % max. 0·5 %
Clay lumps, water-resistant clay	max. 0·0 %	max. 0·0 %
SO_3	max. 1·0 %	max. 1·0 %
Coarse mica (by weight)	max. 0·5 %	max. 0·5 %
Opal, chert, obsidian, calcedony and other amorphous silicates	max. 1·0 %	max. 2·0 %
Lightweight material (specific gravity less than 2·0 g/cm^3)	max. 0·5 %	max. 1·0 %
The total amount of impurities must not exceed	max. 4·0 %	max. 5·0 %

for cement resistance to chemical attack.

The alkali-aggregate reaction test is made on hardened mortar containing crushed pyrex glass graded 0·15–2·5 mm. for sand. The size of test specimens is $70 \times 70 \times 160$ mm. They are water-stored for 28 days. If after this period the sample shows an elongation exceeding 0·05% the cement concerned must not be used with aggregate containing amorphous SiO_2.

The sulphate resistance test of cements is performed with specimens $10 \times 10 \times 30$ mm. made of mortar of dry consistency, which after 28 days of hardening are immersed in an aggressive solution having the same chemical composition as the sulphate-containing water to be analysed. One set contains 12 specimens. The specimens are stored for 6 months in the solution. After this time the samples are tested for bending strength, and the resistance factor of the cement is determined as the ratio of the average value of bending strength of specimens stored in aggressive solution and the strength of control samples stored in well water. The resistance factor should not fall below 0·75.

Chemical analyses of water are governed by a general standard. The present standard specifies only tests of mixing water to be used in cement mortar composed of cement and standard sand in proportion 1:3. The test is made with cubes having a side of 7:1 cm. A control set of the same composition is mixed with well water. The samples are subject to tests 60 days after moulding. The value of the factor of resistance should not fall below 0·90.

TABLE XVII.—PERMITTED QUANTITY OF DELETERIOUS SUBSTANCES IN COARSE AGGREGATES

Deleterious substances	For durable concrete for hydraulic structures, ČSN 73 20 20	For other concretes for hydraulic structures
Loam and clay lumps	max. 0·25 %	max. 1·0 %
Opal, chert and other amorphous silicates	max. 2·0 %	max. 1·0 %
SO_3	max. 0·5 %	max. 0·5 %
Lightweight material (specific gravity less than 2·0 g/cm^3)	max. 5·0 %	max. 5·0 %
Fine impurities (smaller than 0·05 mm.)	max. 1·0 %	max. 2·0 %
Total amount of undesirable material	max. 3·0 %	max. 5·0 %
Flat shaped and elongated particles	max. 15 %	max. 15 %

TABLE XVIII.—AGENTS CONTAINED IN MIXING AND CURING WATER

Component		Allowed quantity	
		Mixing water	Water for curing
Concentration of hydrogen ions	*p*H	Min. 4·5, max. 10·0, for alumina cement max. 8·0	Min. 6·0, max. 9·0, for alumina cement max. 8·0
Floating material of organic nature	mg/l	—	—
Total amount of dissolved matter	mg/l	max. 3,000	max. 1,500
Sulphates	SO_4 mg/l	max. 1,500	max. 600
Organic substances—oxidability	$KMnO_4$ mg/l	max. 50	max. 100
Humus		Yellow colour at the maximum	
Hydrogen monosulphide	mg/l	traces	traces
Ammonia	mg/l	traces	traces

Aggregate tests. The standard test to determine the modulus of fineness is carried out by means of a set of screens specified by the standard ČSN 73 20 21. Complete screen analysis is to be made using a set of screens completed by such screens as would separate fractions from each other. Specific gravity and surface moisture of sand are determined by the syphon method, the vessel having a diameter of 150 mm. and a height of 300 mm. The determination of specific gravity of gravel is carried out on the basis of Archimedes law by weighing a sample of gravel under water level, and after drying its surface by weighing it once again in air. The weight of sample in grams is expected to be 100-fold the maximum size of gravel in mm. The unit weight of aggregate is determined by means of cylindrical containers conforming to Table XX. The aggregate is compacted by vibration.

The percentage of lightweight materials is determined in accordance to ASTM Standard by flotation of aggregate in $ZnCl_2$ or other convenient liquid having specific gravity 1·95 g./cm^3. Approximate determination of total contents of clay, loam and dust in aggregate is made by sedimentation test by volume. Accurate determination of particles having diameter less than 0·06 mm. is made by wet screening on screens with aperture 1 mm. and 0·06 mm. (10,000 meshes per sq. cm.). Accurate determination of clay content is made by sedimentation in a glass cylindrical container, from which a suspension is obtained by mixing the exactly specified weight of 100 cc. of polluted aggregate with water. The suspension is extracted from the container after a period of time corresponding under given temperature to the time of sedimentation of particles having the size of 5 μ on a path of 10 cm. The weight of suspension, obtained as described above, is denoted G. The weight of 100 cu. cm. of distilled water at the temperature of the test is denoted G_1. The clay content in fine fractions in per cent. is given by the formula:

$$P = 3{\cdot}22\ (G_1 - G).$$

The test of aggregates for soundness is made by means of the well-known solution of sodium sulphate. The durability of aggregate is found from the resulting number of freezing and thawing cycles according to classes specified by standard ČSN 73 20 21. The

TABLE XIX.—TESTS ON HARDENED CONCRETE

	Thinwalled cu. m.	Massive cu. m.
Impermeability of surface layers for each	100	600
Impermeability of hearting concrete for each	—	3,000
Compressive strength for each	300	600
Bending strength for each	300	1,000
Durability for each	5,000	20,000

TABLE XX.—TEST FOR UNIT WEIGHT OF AGGREGATE

Maximum particle size	Volume of cylinder	
mm.	litres	
5	5	Diameter: height of cylinder = 1 : 2
20	10	
40	30	
80	50	
150	70	

TABLE XXI.—SIZES OF CONTAINERS

Maximum size of aggregate	Minimum volume of cylinder
mm.	litres
5	5
20	10
40	30
80	50
150	70

shape of particles is determined by individual measurements of grains by means of slide gauges. The percentage of mica content is determined only for sands of 0·15 – 2·5 mm. grading.

ČSN 73 20 24—Tests of Concrete Mix

The standard specifies a test of mixing efficiency by means of determination of variations of specific gravity (free of air) of mortars in a similar way as was used on Grand Coulee dam, Columbia River. In addition to this standard test, it is possible to follow the efficiency of mixing by following the ratio $\frac{\text{water}}{\text{fine particles (less than 0·2)}}$ in samples taken from different places in the mixing drum.

The unit weight is established after vibration of the sample on a vibrating table as soon as complete compaction has been achieved, and is determined by cylindrical containers characterised by the ratio of diameter to height = 1:2. For different values of $D_{max.}$ the containers signified in Table XXI have to be used. The volume of the container must be determined accurately by calibration, the cylinder being filled with water of known temperature and weighed.

The mix proportioning is determined by analysing fresh concrete by a washing procedure on screens and repeated weighing.

The compactness of fresh compacted concrete is determined by the formula:

$$\sigma = \frac{\gamma_{vol}}{\gamma_b}$$

where γ_{vol} = unit weight of fresh treated concrete,

γ_b = specific gravity of fresh treated concrete.

The latter value is to be computed from formula:

$$\gamma_b = \frac{1 + s + g + v/c}{\frac{1}{\gamma_c} + \frac{s}{\gamma_s} + \frac{g}{\gamma_g} + \frac{v/c}{\gamma_w}}$$

where: s, g = weight proportions of sand and gravel respectively on the assumption of the weight of cement to be = 1.

v/c = water/cement ratio.

γ_c, γ_s, γ_g, γ_w = specific gravity of cement, sand, gravel and water respectively.

Consistency of the mix is determined by the well-known Abram's method with the only difference that a large cone (300, 150, 450 mm. height) is used for mixes having a maximum particle size up to 80 mm. The results of tests made with both cone sizes can be converted by means of an experimentally found conversion factor having the value 0·67.

For dryer mixes (slump 0 cm.) the standard specifies consistency measurements by Vebe consistency meter.

The air content in air entrained concrete is determined either by the volume or the pressure method. The device used for the pressure method, specified by the standard, is analogous to the testing devices developed abroad.

ČSN 73 20 25—Testing of Hardened Concrete

Standard ČSN 73 20 25 first specifies curing of concrete specimens and their capping either with cement paste or a mixture of sulphur and clay. Further, it specifies the time when the tests have to be carried out. For simplicity's sake all tests have to be made after 28 days, but it is recommended that the specimens be tested after the so-called "efficient period," by which is understood the time period during which cement exhausts its binding capacity and after which its strength rises only slowly. For slag cement and trass cement the efficient period is believed to be 60 days, for dicalcium (low-heat) cement 90 days. The standard also specifies tests of hardened concrete.

Tests for resistance to adverse chemical reactions are made with test beams 150 × 150 × 450 mm., which have to be immersed in an aggressive solution up to a height 150 mm. At the same time control specimens are to be stored in non-aggressive water. The minimum time of exposure of the specimens to the effect of the solution is 6 months. If the conditions to which the concrete in the structure is expected to be exposed are unfavourable, the tests should be extended to as long as one year. After this time the specimens are tested for bending strength and each of the fractured halves should be tested for compressive strength. From the results of both tests the resistance factor of concrete is computed, i.e., the ratio, compressive strength of solution-stored parts of the sample to compressive strength of specimens kept in non-aggressive water. The factor of resistance of concrete should never fall below values stipulated by standard ČSN 73 20 20.

TABLE XXII.—SPECIMENS FOR IMPERMEABILITY TEST

Maximum size of aggregate mm.	Size of specimens (mm.)			Average area exposed to water pressure mm.
	length	width	height	
40	300	300	120	100
80	400	400	150	100
Over 80*	400	400	200	100

* Aggregate of size over 100 mm. must be removed from the mix.

TABLE XXIII.—NUMBER OF SPECIMENS FOR DURABILITY TEST

Mark of concrete	Number of specimens used for durability tests if the effect of freezing is checked after each	
	50 cycles	25 cycles
T_{50}	3	6
T_{100}	6	12
T_{150}	9	18

TABLE XXIV.—SIZE OF CUBES FOR COMPRESSIVE TEST

Maximum size of aggregate of concrete mix $D_{max.}$	Dimensions of cubes
mm.	mm.
25	100 × 100 × 100
40	150 × 150 × 150
60	200 × 200 × 200
100	300 × 300 × 300

TABLE XXV. VALUES OF FACTOR *K*

Size of specimens	Conversion factor *K*
mm.	
100 × 100 × 100	0·85
150 × 150 × 150	0·90
300 × 300 × 300	1·10

The impermeability test of the concrete is made with square specimens designed according to the pattern given in Table XXII.

The tests are made with a series of three specimens in such a way that hardened concrete specimens (after 28 days) are exposed to water pressure acting in a vertical direction down on an area free of mortar. The initial water pressure of 1 atm. acts 8 hours. After this time the pressure is raised to 2 atm. acting again 8 hours, and so on. In this way the water pressure is successively raised until the pressure value is reached which corresponds to the mark of impermeable concrete (e.g. for concrete V_8 the highest pressure is 8 atm. during 8 hours). The concrete tested is considered as satisfying the required impermeability mark if after this period water failed to leak through at least two of the three specimens of the series. As soon as the test is finished, the specimen is broken apart and the depth of leakage of water is determined and marked with paint. The depth of leakage is measured at three points, and from these measured values the average depth of leakage is computed. To prevent water leaking through the edges of the specimen, these are waterproofed with a watertight cement. If in spite of this measure water leaks through the edges of the specimen in the course of the test, this specimen must be excluded from the test series.

The procedure of comparative tests of impermeability is analogous to the testing procedure described above, with the only difference that water pressure is raised each 8 hours until the moment when moisture appears on the under face of the specimen. This test is designed to compare impermeability of different sorts of concrete.

Durability tests of concrete. Durability of concrete is determined by alternating freezing and thawing. The number of specimens to be used for the tests is given in Table XXIII. After 28 days of hardening (or after the efficient period of hardening) the samples are saturated with water for four days so that water reaches first the level of $\frac{1}{4}$ the height of the specimen, After 24 hours water is added so as to reach the level of $\frac{1}{2}$ the height of the specimen, and after 72 hours the specimens are completely submerged, so that a 2 cm. depth of water is above the top of the specimens. After 96 hours the samples are taken out, their surface dried and the specimens are weighed with an accuracy of 0·2%. The average value of weight receives the notation *A*. After weighing, the specimens are immediately put into a refrigerator, where they are stored in a temperature of $-17°C$. Starting with the moment when the surface of the specimens is cooled down to $-17°C$., the specimens are left in the refrigerator for 4 hours. Then they are taken out and put into a water bath having a temperature $+20°C. \pm 2·5°C$. In this bath the specimens are kept for a further 4 hours. The same cycle is then repeated.

After each 50 (25) cycles, the surfaces of the specimens are dried, then the specimens are weighed (weight *B*) and one series of specimens is tested for bending strength. From the data thus obtained the proportion of weight loss in per cent. is calculated from the formula:

$$G_n = \frac{A - B}{A} 100.$$

Similarly the comparative series of specimens, water-stored as in the tests described above, are tested after the equivalent period of hardening. This period is obtained from formula:

$$T_c = a + 0·2\ n,$$

where a = period of hardening in days previous to freezing test,

n = intended number of freezing and thawing cycles.

The factor of frost resistance is computed as a ratio of the average values of bending strength of specimens subject to alternating freezing and thawing and comparative specimens. The factor of frost resistance must never fall below 0·75 or the weight loss after a given number of freezing and thawing cycles must not exceed 5%.

Test for compressive strength. The test for compressive strength is made on cube-shaped specimens, the size of which depends on the maximum size of aggregate. This is done to eliminate the necessity of screening or separating grains for bearing concretes. The size of cubes with respect to the maximum size of aggregate is determined from Table XXIV. Aggregate pieces exceeding 100 mm. must be separated from the mix. The specimens have to be made in a way analogous to the method by which concrete on site is produced. If the concrete is compacted by ramming, the specimens should be compacted in the same way; if the concrete is compacted by vibration,

(Continued on page 424)

Experiments on Surge Waves

This article describes experiments which were conducted to ascertain whether the profiles of various types of surge waves recorded in a model channel corresponded with those deduced analytically. The electronic recorder used has already been described in this journal*

By J. A. SANDOVER, Ph.D., B.Sc., D.I.C., A.C.G.I., A.M.I.C.E., and O. C. ZIENKIEWICZ, Ph.D., B.Sc., D.I.C., A.C.G.I., A.M.I.C.E.

WHEN a rapid change of discharge occurs at some point in the length of an open channel, disturbances apparent on the surface of the fluid are created. These tend to move away from their originating source with a certain finite velocity and are termed "surge waves" or "gravity waves of translation." The size and shape of such waves depends on the magnitude and type of the change which has initiated them. The object of the experiments to be described was to study the profiles of certain types of such waves and to compare them with profiles obtained from theoretical considerations.

Broadly, surge waves can be divided into two categories—positive and negative. In the former, the average depth of the surge front is always greater than the initial depth, while the reverse is true of the latter. In addition, surges can move either in the downstream or in the upstream direction. Thus there are four basic types of surge waves possible and these, in some cases, where reflections at the channel ends occur, may become superposed, resulting in a much more complex picture.

In practice, surges may be caused by numerous circumstances. For example, when a turbine is put into operation in a power station, positive surges moving downstream will be created in the tailrace channel and negative surges moving upstream in the supply channel. A sudden stoppage on load rejection is similarly followed by negative waves in the tailrace and positive ones in the supply channel. An abrupt opening of lock gates or of a sluice will cause positive waves in the canal and negative in the lock, while a sudden closure of a sluice gate in a channel in which water is moving will be associated with reverse phenomena.

In natural water courses close to a tidal estuary positive surges moving upstream are frequently observed. These are called bores or "aegers" and are caused by a rapid build-up of the rising tide in the sea.

When the change of flow is sufficiently rapid, the profile of the head of the disturbance reaches a semi-permanent profile which gradually decreases in magnitude with its travel along the channel owing to friction and ponding effects. In the investigation described, attention will be focused on positive surges moving downstream in stationary water. These, depending on the relative magnitude of the flow increase, can take on an undular, undular-breaking or steep-fronted form as illustrated in Fig. 1. The latter form of these three resembles in essence the frequently encountered hydraulic jump and indeed can be shown to obey the same laws. Negative surges exhibit the types of profile shown in Fig. 2, but with no equivalent of a steep-fronted surge.

The object of the experiments was to record the profiles of a number of surges similar to those of Fig. 1(*a*) and (*b*). As undular surges of this type are most frequently met in practical problems, their study is of some importance and the understanding of the phenomena involved is essential for the successful design of many components of hydro-electric projects. Fig. 3 shows a surge wave in a model channel.

Historical Note

To date, no exact analytical solution to the problem of undular surges is available, and all approaches to the problem must, by necessity, depend on a number of simplifying assumptions. The well-known classical theories concerning the somewhat simpler problem of periodic waves prove, however, to contain much of the information required. In this connection, mention must be made of the work of Boussinesq.

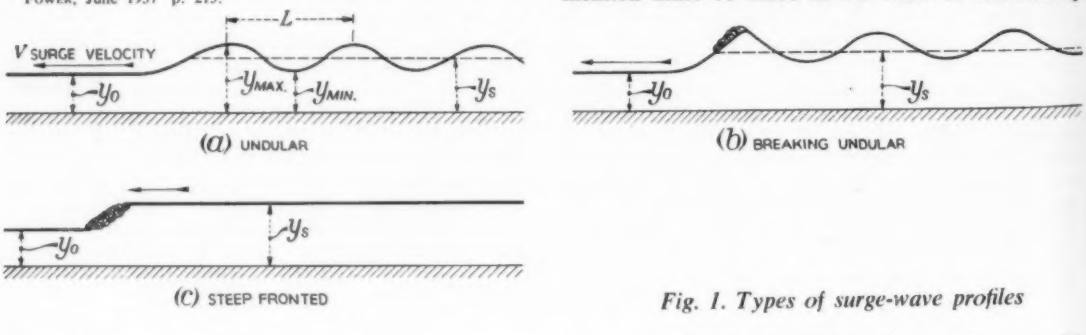

Fig. 1. Types of surge-wave profiles

* "An Experimental Wave Recorder," by J. A. Sandover, WATER POWER, June 1957 p. 213.

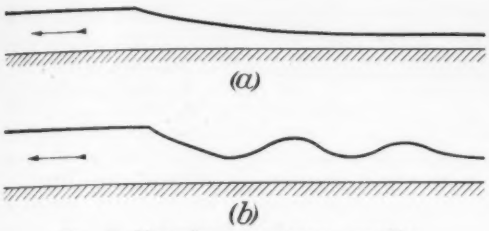

Fig. 2. Negative surge-wave profiles

Rayleigh, Stokes and Korteweg and de Vries[7].

In the same epoch in which these notable advances on the theory have been made, some extremely interesting and useful experiments were carried out by Scott Russell[11] and Bazin[1]. Scott Russell made his first observations on surge waves created in canals by the opening of lock gates and he succeeded in following such waves (on horseback) for distances of several miles. Later, in a small laboratory flume, he conducted a classic series of experiments on the solitary wave —a phenomenon in many ways akin to that of an undular surge wave. Bazin undertook his work in a much larger model channel (about 1 metre deep and 600 metres long) in which he succeeded in measuring with reasonable accuracy some of the salient features of undular surge wave trains.

Little attention was paid to the problem until Favre[5] undertook his well-known investigations. Although the major part of his work is concerned with the broader problem of unsteady flow in open channels, he made some very careful measurements of several undular surges. In recent years Binnie and Orkney[3] conducted a very illuminating investigation into the problem of stationary wave trains which in many respects resemble the undular surge profiles with which this paper is concerned.

A renewed effort to solve the problem theoretically is apparent in recent years. An approach developed by Lémoine[8], although open to criticism (as all friction effects are ignored), does provide a good first approximation. Subsequent work by Serre[11] and Benjamin and Lighthill[2] shows that friction effects are of prime importance in the problem. Although in many respects identical results are obtained by the last authors mentioned, Serre's approach appears to permit a determination of actual surge profiles by a step-by-step procedure. It is, in effect, his method which has been extended by the present authors[13] to obtain the detailed solutions referred to later.

N.B. The numbers refer to the Bibliography.

Apparatus Used

For the sake of simplicity in the calculations, a channel of rectangular section, with a horizontal bed, was considered. A model was made, 48 feet in length, 5 in. wide and 10 in. deep internally. The inner surface was completely lined with glass. This was held in position by a metal frame, the base of which was a 6 in. by 3 in. mild-steel channel. The glass floor of the channel was bedded on a black putty-like substance (trade name—"Black Magic"), which was also used for the side joints, in a manner similar to that used in aquaria. The channel was supported on threaded bolts, thus enabling it to be raised or lowered about the inlet end for future experiments with variable slopes. It was supplied at the inlet with water by means of a screw valve and slide valve. The slide valve could be opened almost instantaneously, allowing the water to flow into a large tank of the same internal width as the channel. It was hoped that this tank would help to reduce the turbulence of the supply, and this design proved successful. The inflow was measured by an orifice meter connected to a mercury

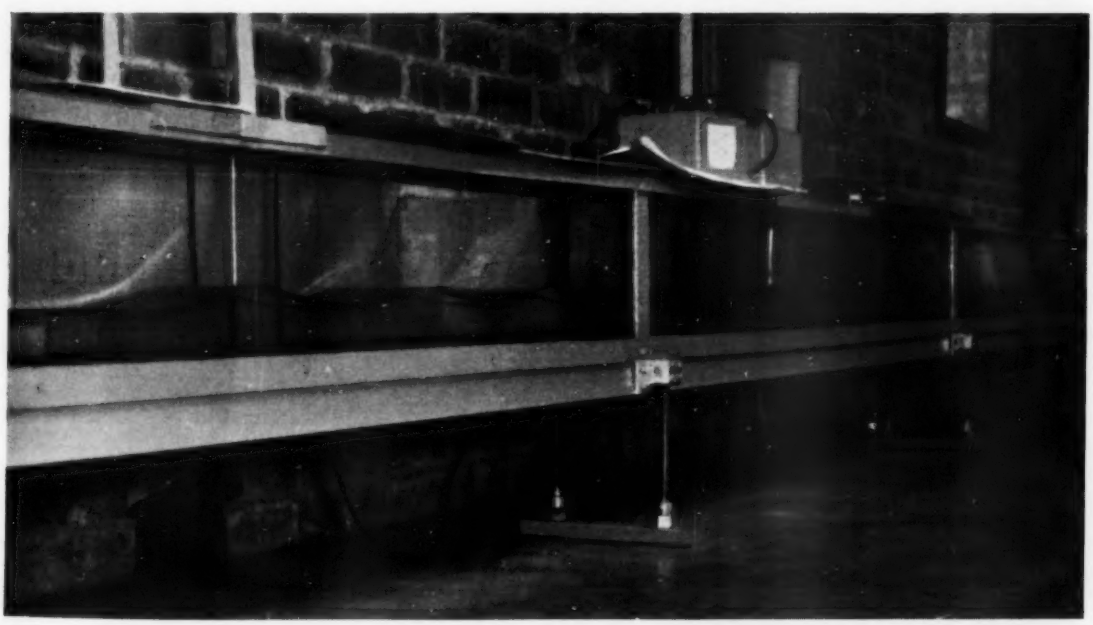

Fig. 3. Detail of channel showing surge wave and depth gauge

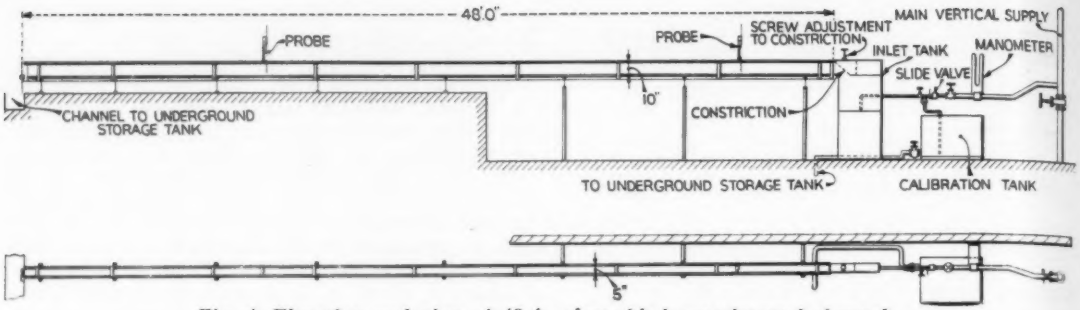

Fig. 4. Elevation and plan of 48 ft. glass-sided experimental channel

manometer. At the other end of the channel were two gates. The first was of the guillotine type and could be shut completely at any desired instant, or partially to any required extent. The second gate was of the weir type operated by a rack and pinion. After leaving the channel the water passed to the laboratory storage tanks, and was then pumped to a tank in the supply tower. This ensured a circulation of the water which had been treated with sodium nitrite to inhibit the formation of rust. A point gauge, Fig. 4, was employed to check the initial depth in each experiment.

The weir gate could therefore be set to give any desired initial depth of water in the channel (checked by the point gauge), the screw valve adjusted to give any desired discharge into the channel and finally, the slide valve released this discharge into the still water in the channel thus creating a surge wave.

Recording Apparatus

Once the surge was obtained, the problem of recording its profile had to be overcome. This was achieved by using a "capacitance type, depth gauge." The concept of using a varying capacitance as a measure of varying depth is not novel. The basic principle of this method of measuring depths is ingenious and simple. If the water and a piece of metal can be used as the two conductors of a condenser, then as the

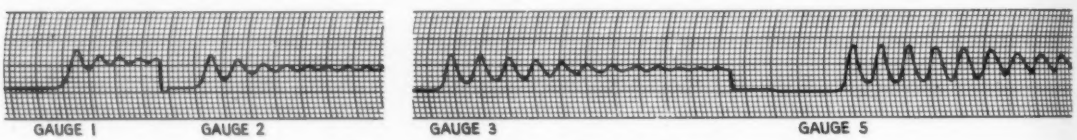

Fig. 5. Record chart of a typical run

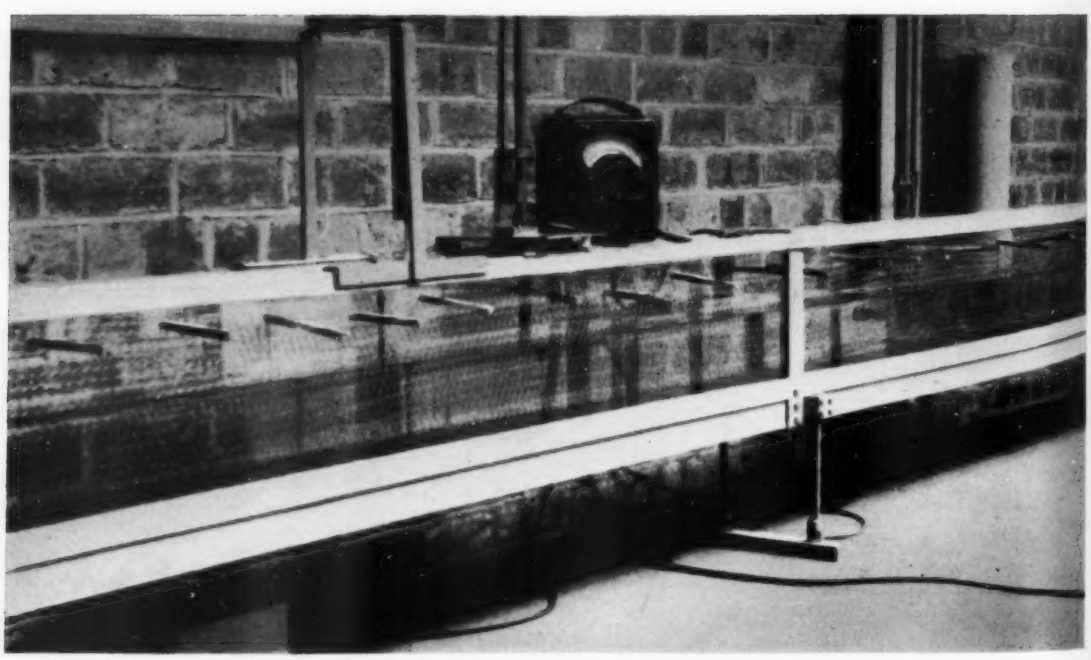

Fig. 6. Channel lined with wire netting

water level varies, so the capacitance of the condenser will vary. This varying capacitance can then be interpreted as a varying voltage, which can be recorded.

In practice this idea needs considerable elaboration before accurate results can be obtained. The condenser was formed with a length of small diameter P.V.C. covered cable, tensioned between two perspex blocks, held apart by thin strips of spring steel. The water and the filament of the cable act as the condenser plates, and the P.V.C. as the dielectric. The end of the cable that remains under water has to be carefully insulated. It would have been easier to have doubled the cable, then both ends would have been out of the water; however, the total capacity range of the available electrical apparatus would not allow this. For the same reason, very thin enamelled wires, which would offer less resistance to the water could not be used.

The lower perspex block of the frame fitted into a groove in the channel bed, cut in a brass strip inset between the glass panels. There were eight of these grooves along the length of the channel, but only six were in use at any one moment. Thus recordings at six separate representative positions could be obtained as the surge moved down the channel.

The depth gauge was connected to an oscillator circuit working near 2,000 m.c.p.s. As the depth varied, the capacity varied also, and this caused the oscillator's frequency signal to alter. This was fed into a discriminator circuit where the change in signal frequency was transformed into a varying d.c. voltage. The output could be recorded on either a cathode-ray oscillograph, or as in the experiments to be described on a pen recorder *via* an amplifier. As only one pen recorder and amplifier were available, the d.c. voltages were fed to the amplifier through a manually operated rotary switch[9].

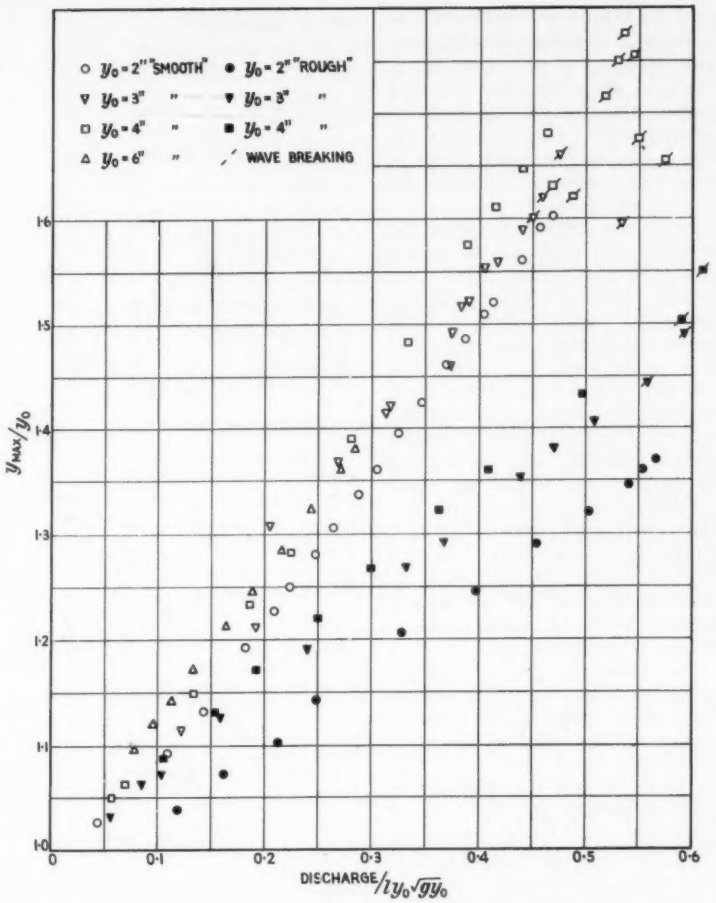

Fig. 7. First wave height-initiating discharge

Calibration of Depth Gauges

It was found that the effect of raising and lowering the water level gave identical results on the recording apparatus with those obtained by lowering and raising the depth gauge. This gauge movement was effected by means of a rack and pinion. A scale was clipped to the rack reading against a vernier. The depth gauge was therefore calibrated by raising and lowering it rather than by changing the water level.

All the recordings were made on a pen recorder; fitted into the mains electrical supply to the recorder was a long cable with an on-off switch at the end. Thus the recorder could be operated from any one of the depth gauges.

The change in elevation was then plotted against the pen deflection of the recorder to give a calibration curve for each depth gauge. In all cases the result was a straight line.

To ensure that the depth remained constant throughout the calibration, the guillotine gate was shut and a considerable time was allowed to ensure that the water surface remained steady.

Forming and Recording a Surge Wave

The weir gate was set to give the required initial depth. The depth gauges were returned to their slots, and the guillotine gate removed. After an interval of about fifteen minutes to allow the water level to settle, the electrical apparatus was reset to give zero output for this depth, and the rotary switch set to the first depth gauge.

The screw valve was set to give a small discharge, and the slide valve opened rapidly. As the wave approached each gauge, the record from the gauge was switched to the pen recorder by means of the rotary switch. At the end of the experiment, the recorder was stopped, and the reading of the calibrated manometer taken. This was repeated with increasing discharges until the first wave broke. Each experiment was called a "run," and each set of runs at a particular initial depth was called a "series." Four "series" of runs

were made at initial depths of 2, 3, 4 and 6 in. An example of the record of a run is shown in Fig. 5. Then to determine the effect of friction, the channel was lined internally with ½ inch wire netting, Fig. 6, and another three series of runs was made at initial depths of 2, 3 and 4 in.

From the calibration curves the depth at any point on the record could be determined. From the record, knowing the distances between the depth gauges and the speed at which the recorder paper was passing the pen, the mean velocity of the head of the surge between the gauges could be determined. If this was plotted against distance for each run, the velocity at each gauge could be interpolated or extrapolated. This was then plotted as velocity at each gauge against discharge. Assuming this velocity to be correct, the wave profiles of the undulations forming the surge could be determined in each position.

As both the profile and the velocity of the wave are subject to a slow variation, profiles obtained by the method in which depth at a fixed point is recorded in time are not the true instantaneous profiles of the wave. A comparison of simultaneous recordings, obtained by this method and from photographs, however, reveals agreement which is well within the limits of experimental error.

A final series of runs enabled some estimate to be made of the effect of the time of opening the slide valve on the shape of the surge.

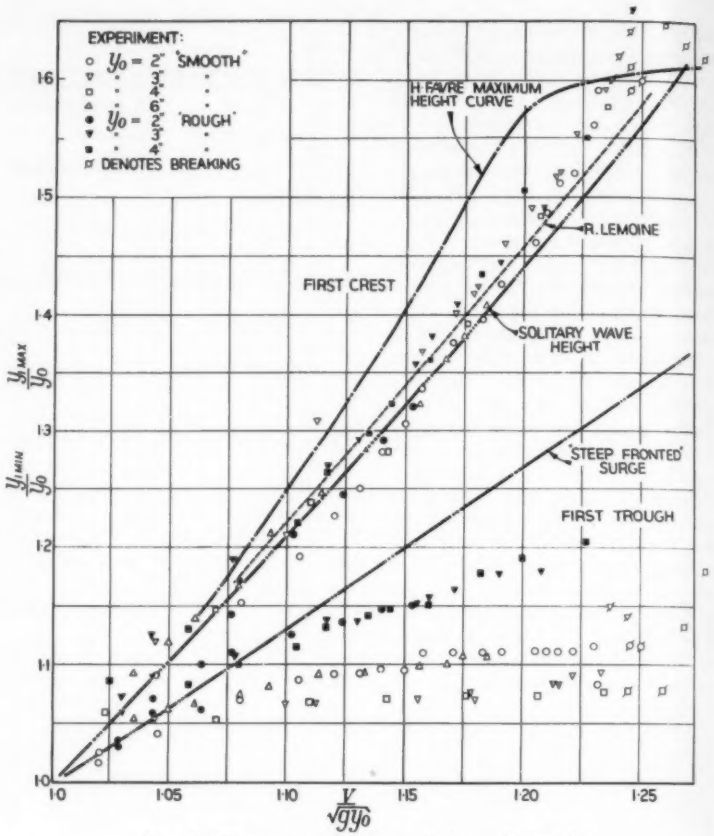

Fig. 8. First wave height-velocity relationship

Experimental results

It was immediately obvious that the surge did not appear fully formed at the inlet. The waves formed one after the other as the surge progressed downstream. Therefore, the further the surge progressed the larger the number of waves in the train. As the surge leaves the inlet, the first wave starts to build in amplitude. After a short distance this wave reaches a maximum amplitude, which thereafter decreases very slowly. The waves formed successively after the first go through similar changes, but none attains such a crest height as the first at its maximum. Eventually the leading waves reach a relatively stable shape but they steadily decrease in size. Each crest is a little higher than the one in front, owing to the mean slope of the water surface, but the amplitude of successive waves decreases.

Quantitative discussion of comparison of the profiles can only be carried out with profit when the final semi-permanent profile is developed. Although profiles were recorded at several points in the channel, numerical data referred to later are almost invariably taken from the records at gauge No. 5, which was situated some 40 ft. from the inlet end of the channel.

At a particular gauge, as the discharge increased, the wave length decreases, and the crest height increases until breaking occurs. Along the length of the channel, however, for one run the wave length increases at first then steadily decreases. This sequence of events occurs more rapidly at the smaller initial depths, when the effects of friction are comparatively more marked.

At low discharges the wave profiles were almost sinusoidal, but as the discharges become greater the shape approximated very closely to a "cnoidal" form. In this form the crests are much more narrow and peaked, while the valleys are relatively long and rounded. At the point of breaking the wave becomes cusped at the peak.

It is interesting to note that when the breaking of the first wave of the surge has just started, the general sequence of the growth of the wave is somewhat different from that previously described. The wave on reaching the breaking height spills and then re-forms at a smaller height without breaking; the height then steadily increases until breaking is resumed again. This phenomenon explains the slightly erratic heights of waves recorded near to the breaking point.

When discussing the phenomena on a quantitative basis, attention must be focused on some easily measurable characteristics. For the purpose of comparison, the variables considered shall be the height of the surge at its first crest and at its first trough, and also the first "wave length." The first crest height

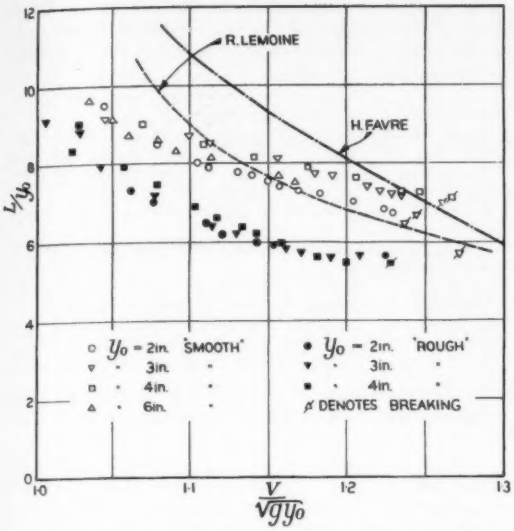

Fig. 9. First wave length-velocity relationship

is of importance, as it approximates closely to the maximum height of surge. The wave length considered is in general only very slightly longer than those succeeding it.

Fig. 7 shows a plot of the first wave crest height against the initiating discharge on a non-dimensional basis. Similar curves could have been plotted for the other two variables. It is at once evident that no correlation of results is possible on this basis, the friction effects obscuring entirely any similarities present. As the first obvious effect of friction is to create a general slope of the water surface beyond the head of the surge which results in a lower effective discharge at the head of the surge than at the intake, it was found more expedient to plot the results on the base of the velocity of the surge wave. Thus, the only effects of friction would be those directly affecting the profile. In Figs. 8 and 9 the crest and trough heights as well as the wave length are plotted, again on a non-dimensional basis. A very remarkable correlation becomes at once apparent for the first wave height. Friction appears to have very little effect on this, whereas the trough height and the wave length show respectively an increase and a decrease with increasing friction effects.

Although the above form of plotting has not been used in the papers referred to, the results obtained experimentally by Favre and theoretically by Lemoine can easily be superposed. It is seen that the latter results form quite a good approximation. Results obtained by the simple hydraulic jump formula—

$$\frac{V}{\sqrt{g y_o}} = \sqrt{\frac{1}{2}\frac{y_{max.}}{y_o}\left(1+\frac{y_{max.}}{y_o}\right)} \quad \ldots (1)$$

shown also on Fig. 8—are obviously not applicable within the range considered, although the rule noted by Bazin, that the height of undulations above the still water level is some 50% greater than the height of the hydraulic-jump type of surge, is not far from the truth.

Breaking of the wave first commences when the value of $V/\sqrt{g y_o}$ is in excess of 1·23, a result in good agreement with the figure of 1·25 quoted by Favre.

A few experiments were also undertaken to estimate the effect of the time of opening of the slide value on the formation of the surge. Up to an opening time of 2 secs. the effect was discernible but extremely slight. At 3 secs. the effects were larger; at 5 secs. the effects were very noticeable, and at gauge 2 the changes in the crest height and wave length were −50% and −17% respectively. At gauge 5, however, the effects were much smaller being −5% and −8% respectively. The change in valley depth varies from +35% at gauge 2, to +60% at gauge 3, and +30% at gauge 5. Gauge 2 is 10 ft. from the inlet.

Theoretical Results Compared with Experiment

At this point some reference must be made to the theory governing the phenomena. Space limitation prevents the deduction of the necessary equations. These can be found in the works already referred to [1] and [2] and also in a recent paper by the writers[13]. The reader will have to accept without proof the following, approximate, governing equations:

$$\frac{V^2}{6 g y_o^2}\left(\frac{dy}{dx}\right)^2 = Hy^2 - My - \frac{y^3}{2} + \frac{V^2}{2 g y_o^2} \quad \ldots (2)$$

$$\frac{dH}{dx} = \frac{f}{2g}\frac{V^2}{y}\left(1-\frac{y_o}{y}\right)^2\left(1+\frac{2y}{l}\right) \quad \ldots (3)$$

$$\frac{dM}{dx} = \frac{f}{2g}V^2\left(1-\frac{y_o}{y}\right)^2\left(1+\frac{2y}{l}\right) \quad \ldots (4)$$

in which x and y are the depth of water at any point and the horizontal distance respectively. V, as used before, stands for the wave velocity, 2 the channel width, and f is Darcy's friction coefficient. H and M are energy and mometum functions, which at the starting point of the problem, i.e. where the depth is constant and equal to y_o, have the following values:

$$H = y_o + \frac{V^2}{2g} \quad \ldots (5)$$

$$M = \frac{y_o^2}{2} + \frac{V^2 y_o}{g} \quad \ldots (6)$$

It can be shown readily that if friction is non-existent then the only non-trivial solution to the above equation system results in the classical solution of the solitary wave profile in which the maximum depth is given by:

$$\frac{y_{max.}}{y_o} = \frac{V^2}{g y_o} \quad \ldots (7)$$

This simple result is plotted in Fig. 8 and gives a very good agreement with the heights of the first undulation.

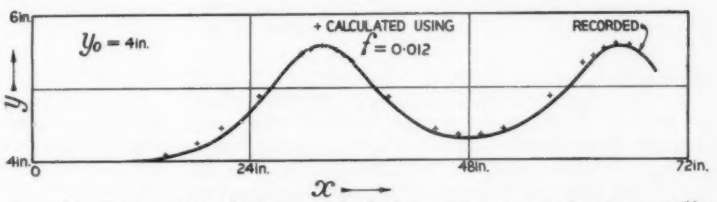

Fig. 10. Comparison between calculation and measured wave profiles ($V = 1{\cdot}172\sqrt{g y_o}$; smooth)

By a tedious step-by-step integration it is possible to integrate the equations for any specified value of f. Several such integrations have been carried out and compared with carefully measured test runs. A typical comparison is shown in Fig. 10. In these calculations special attention had to be given to the value of the friction coefficient assumed. This obviously has to be larger than in a steady flow case as the boundary layer is very thin at the head of the surge. The values used were derived from the consideration of the average slope of the head of the surge and proved to give very close comparisons.

In the above numerical work the general trends already observed in the experiment were borne out. The very slight effect of friction on the first wave height, the increase of the trough height and a decrease of the wave length with increasing values of f were noted again.

It has been remarked that certain of the results were confirmed by photography. This was found to be a very accurate and reliable method for a limited number of recordings of single undulations. A grid was marked on a sheet of paper in Indian ink and this was carefully glued to the glass side of the channel. The photographs were taken with the aid of "Stroboflood" lamp, and the record could be read to within $\pm 0{\cdot}01$ in. using a travelling microscope.

When compared with the results obtained with the pen recorder it was found that the crest heights agreed within very small limits. It was estimated that the errors in these recorded heights would be of the order of $\pm 0{\cdot}015$ in. The errors were greater at the troughs of the waves, as the meniscus effects were largest, and it was estimated that the errors would be of the order of $-0{\cdot}01$ to $+0{\cdot}02$ in. The recorded wave lengths were found to be accurate to within $\pm 0{\cdot}25$ in. This would appear to be considerable, but in fact gave rise to a maximum error of approximately 3%.

Conclusions

While the design of the channel and choice of recording equipment posed certain problems, probably the greatest single difficulty was the design of the depth gauge. When, after many trials, a gauge was designed which fulfilled all the conditions imposed, the remainder of the experimental work was considered to be a relatively simple matter. It is hoped, in future work, to use a number of refinements that will reduce the work in interpreting the results. Further, it is hoped to extend the scope of these investigations to the more difficult problems of channels with sloping beds and variable cross section.

From the practical point of view, the work outlined should permit at least an approximate determination of the maximum heights of surge waves without recourse to model work. Although the methods of doing this have not been discussed explicitly here, it is found that provided the average flow at the head of the surge is known, the surge velocity can be computed reasonably accurately from the "hydraulic jump" type of formula. The maximum heights of the undulation can then be found by the simple "solitary wave" expression. With the growing use of underground power stations in hydro-electric work, and the consequent use of long tailrace channels, the importance of a more exact understanding of the surge wave problem is obvious.

The work was undertaken in the Sanderson Engineering Laboratories of the University of Edinburgh, and the authors wish to thank Professor R. N. Arnold for the facilities afforded.

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the specimens have to be vibrated on a standard vibrating table until perfect compactness is reached.

Compressive strength of concrete is related to specimens $200 \times 200 \times 200$ mm. If the strength was tested on specimens of different size, the test results must be multiplied by a conversion factor K, the values of which are given in Table XXV. The test is carried out with a series of three specimens. If any of the resulting values differs by more than 20% from the arithmetic mean, this specimen must be eliminated from the total number and the result must be calculated as the average of the remaining test results.

Bending strength of concrete. The specimens for this test have the shape of prisms with a cross-section of 150×150 mm. and a length of 1,200 mm. The mix samples are vibrated until complete compaction is reached. The specimens are loaded by two loads operating at a distance of 200 mm. $\pm$ 1 mm. from each other, the span being 1,000 mm. $\pm$ 1 mm. The bending strength of the concrete is computed from the following formula:

$$\sigma_{Po} = \frac{120\ P_{\max.}}{bh^2}$$

where σ_{Po} = bending strength, kg./cm^2.
$P_{\max.}$ = failure load, kg.,
b = width, cm. (accuracy 1 mm.),
h = height, cm. (measured at the point of failure, accuracy 1 mm.).

The compressive strength is then determined on both parts of the broken prism as a checking value.

Conclusion

The standards describe further in detail the testing procedure and equipment used. Although some of the requirements of the standards are somewhat severe, especially as to the properties of materials used, the possibility is given to test concrete components in the concrete itself and these tests are then decisive.

The standards described and reviewed above are tentative for the time being, as their conception is in many respects rather new in Czechoslovak practice, and it is necessary to find time and opportunity for the workers in construction practice to change their minds and habits and to research workers to supply sufficient verification data.

The IAHR Congress

A commentary on the Seventh Congress of the International Association for Hydraulic Research held in Lisbon in July

PART TWO

THE sections on intakes and associated works and on free subjects were so strongly suppported that we cannot do more than make a few brief comments on those papers more directly within our purview. As we intimated last month, although these two sections accounted for about four-fifths of the total number of papers they did not provide a focus of controversy as was the case with the sections on scale effect and on cavitation. This is not to imply that the papers lacked quality; indeed, many of the papers were of more than ordinary interest, and some were concerned with absolutely original work. Furthermore, despite this seeming lack of a central problem for discussion, in both sections a process of natural classification could be observed, which provided some degree of co-ordination, and which we can conveniently follow in our commentary.

Intakes for Hydro-Electric Schemes

Within the section "Intake Works, Tunnels and Channel Outlets," which comprises 25 papers, we will consider a group of papers describing the hydraulic design of intakes for actual hydro-electric schemes. One of these, by I. W. McCaig, of H. G. Acres & Co. Ltd., describes the design and model testing of the intake for the Bersimis Lac Casse development.* Preliminary tests were made with a Johnson tube-type intake using a tapered metering slot, but although an even draft was obtained entrance losses were high. The intake was therefore redesigned using streamlined piles to accelerate and turn the flow coming from the trashracks so that it merged smoothly with the flow in the tube. Minimum hydraulic losses automatically occurred when the draft through all trashracks was nearly equal.

Model tests on the intake design for the Caroni (Macagua No. 1) scheme, Venezuela, which were carried out at the Hydraulics Research Station, Wallingford, are discussed in a paper by H. D. Morgan and J. S. Burgess. The scheme utilises a largely natural head of 35-40 m. Initially four generating sets of 62 MW capacity are being installed, each taking a maximum normal flow of 185 cu. m. per sec. The intake is being constructed at the head of natural falls with the power house immediately downstream in a deep cut. For maintenance and emergency closure of flow, a gate is required at the head of each penstock pipe, and after consideration of the range of useful headrace water levels, the intake was designed to utilise 10 by 11 m. tainter gates, designed to close automatically against runaway turbine flow. Gate closure and steady flow conditions were studied on a one-gate $1\frac{1}{2}$ in.: 1 m. model and found satisfactory.

* This development was described in WATER POWER, June 1957, p. 203, and July 1957, p. 243.

An interesting problem arose in connection with the Eucumbene-Tumut tunnel in the Snowy Mountains scheme, and is discussed in a paper by J. W. Ball and J. C. Schuster of the U.S. Bureau of Reclamation. About 10 miles from the Adaminaby portal an 18 ft. vertical shaft receives water from a small diversion reservoir at Junction Pond and discharges it to the 21 ft. tunnel 300 ft. below, where the flow may be in either direction. Three shaft designs were considered: one in which the water entered through a morning-glory shaped entrance and plunged into the partly full shaft, one in which the jet from a control at the top of the shaft plunged through a vacuum in the upper portion of the shaft into the partly full shaft, and one in which the shaft was kept under pressure by a control at the bottom of the shaft so that air would not enter. The considerations and model tests which concerned each design and led to the final hydraulic features of the third and adopted design are described in the paper.

Swedish practice in intake design was represented by two papers. One by S. Angelin of the State Power Board refers to model tests on intakes for some 15 stations built by the Board, and advances some general rules for the location of an intake itself, and the effect of the penstock. Faulty intake design leads not only to loss of head but also to a loss of turbine efficiency and to an increase in the cavitation characteristics of the turbine. The other paper, by E. Reinius, of Chalmers University of Technology, Gothenburg, refers to systematic model studies comprising 100 alternatives, as a result of which an optimum design in respect of head loss and construction cost is proposed.

Two papers were also presented by Japanese authors. One by T. Sakamoto, of the Central Research Institute of the Electric Power Industry, describes model tests to prevent vortex formation in the intakes to Koshikata and Niyodogawa power stations. The other, by K. Tanaka, of Osaka University, deals mainly theoretically with the improvement to the supply-canal inlet to Uji power station.

A particularly interesting paper by Mm. Labaye and Molbert, of Electricité de France, describes the design of small gravel-trap intakes used for high-altitude stream collection. To be economic such intakes must operate automatically with the minimum of maintenance, the fewest of visits by supervising staff, and with low water losses. Two designs are described, one in which gravel disposal is determined by a hydraulic "clock" and the other incorporating an automatic gravel detector. Most of the intakes of the Roselend project will be provided with gravel traps to these designs.

To deal now with somewhat more general questions, D. F. Denny and G. A. J. Young gave an

account of the useful work they are doing at the British Hydromechanics Research Laboratory on the prevention of swirl and vortex formation at intakes. As we have recorded previously, most of their investigations have related to the design of pump sumps, but they have more recently concluded a valuable series of model tests on the intakes for Kariba.

An analysis of positive surge waves in power-station supply channels was presented by J. Cábelka and P. Gabriel of Czechoslovakia. A comparison is made between computations and measurements of the actual behaviour of a surge wave in the channel of a power station, and a simple and quick method of computing positive surge waves is propounded.

Diversion Tunnels

Three papers were submitted on diversion tunnels. E. Reinius, whom we have already mentioned, presented an unusually interesting one on model tests of the diversion tunnel of the Höljes power plant on the Klara river. The tunnel has to transport not only water (830 cu. m. per sec.) but also timber and ice on the water surface. The width and depth of the tunnel, and also the radius of the tunnel, were designed so that the rock walls need not be surfaced with concrete and yet permit the passage of logs through the tunnel without risk of log jams forming across it. The tunnel inlet was designed to allow the floating pieces to pass a hydraulic jump which had a surface stream on top of a bottom roller, and the tunnel outlet was designed wide and shallow and with retarding velocity in order to lessen erosion in the river bottom downstream of the dam.

An interesting design of diversion-tunnel outlet, which is to be used in the construction of the Portuguese Miranda station on the International Douro, is discussed by D. Pinto da Silva of Hidro-Eléctrica do Douro. The cost of diversion works can often be considerably reduced by recovering as much as possible of the velocity head at the outlet. Progressive widening of the tunnel outlet section, when possible, solves this problem, but as this was difficult to apply at Miranda an exterior transition channel was designed, the channel dimensions being calculated so that the maximum flow in the terminal section was near to critical. Model tests confirmed the values obtained by computation.

The third paper in this group was presented by T. Ito and K. Ashida, of the Public Works Research Institute, Ministry of Construction, Japan. This paper deals, largely mathematically, with the hydraulics of diversion tunnels, and particularly with the hydraulics of transitions.

Silting at Intakes

Several papers were concerned with the problem of silting at intakes. A contribution by H. Villatte describes methods developed on scale models at the Sogreah Laboratory for the protection of intakes from silt entry. These methods comprise: (1) the use of a trench, acting as a stone trap, placed at the foot of the trashracks with a wall forming a weir along the river side of the trench; (2) the use of a trench bounded on the river side by a weir crest provided with a transverse groove from which the material is removed by a separate system; (3) transforming the sill into a hollow dyke by means of traps in the upstream face, thus removing the bed-load under pressure flow from the foot of the trashracks; (4) combining trench and retaining wall with a hollow dyke; (5) retaining the bed-load until it is inside the headrace canal and then making use of the effect of flow round a bend to remove it through an orifice placed at the point where the bed-load concentrates; and (6) using floating flow guides which divert the bed current away from the intake.

A theoretical study is made by M. Vojinovic, of the Avala Hydraulic Laboratory, Belgrade, of the parameters influencing the streamline pattern near the intake and the silt entrainment into it. It has been shown by tests that the motion of water and silt near the intake zone depends on a great number of parameters. The silting of the intake diminishes with the decrease of the intake width, bed roughness, and river and intake discharge, on the one hand, and with the increase of the river depth and width, on the other hand. This holds only if one of the parameters varies and the others remain constant. There are several ways of preventing the silting of the intake, the most efficient ones being the structures creating a helicoidal motion near the intake zone. The direction of this motion should be similar to that on the concave side of the river bend. The effects of several structures on silting have been investigated by model studies. A further interesting contribution from Belgrade, by M Boreli of Belgrade University and S. Bruck of the Hydrotechnique Institute, pointing out that in alluvial rivers with a natural branch, the sediment load is often much smaller in the branch than in the river itself, so that it becomes attractive to locate an intake on the branch.

A method of calculation for rectangular settling basins below intakes is advanced in a paper by J. Pantélopulos of the Hydraulic Research Institute, Prague.

Turning now to one or two miscellaneous papers under the heading of intakes, J. Megard, of the Sogreah Laboratory, gives some examples of the problems presented by air vents, particularly in the headrace tunnels of power stations. Two papers are concerned with trashracks. One by L. Levin of Société des Forges et Ateliers du Creusot studies the hydraulics of trashracks with special reference to damage sustained in practice, and the other, by L. Castex of Laboratoires d'Hydraulique de Toulouse, considers the phenomenon of grid vibration and proposes a new type of screen designed to avoid trouble from this source. Two other papers deal with head losses in tunnels. J. Faure, of Laboratoire National d'Hydraulique de Chatou, gives an account of an experimental investigation of losses in partly lined supply tunnels, paying particular attention to the transitions, and F. T. Mavis, of the Carnegie Institute of Technology, Pittsburgh, and F. Bustamante, of the Shell Oil Company of Venezuela, report on a basic model study simulating effects of air entrapment in unlined water tunnels through rock. It was found that there was no difference in effective roughness of a straight conduit whether the roof pockets were filled with water or air.

Flow Problems

In the section "Free Subjects," comprising no less than 42 papers, the process of natural classification

to which we have already referred, can again be observed, and we will commence our summary with an account of a group of papers dealing with problems of flow.

A study of flow conditions at the inlet to Francis-turbine draft tubes is described by P. Campas of Electricité de France. A technique has been developed by which it is possible to check the similitude between model and prototype, to choose the best form of draft tube with greater speed and certainty, and to choose the profile and diameter of the runner outlet so as to minimise the draft-tube losses and avoid an efficiency curve steeply drooping on overload. In fact, the performance of any runner can be assessed with a given draft tube, and of any draft tube with a given runner.

The measurement of flow in a circular conduit by the salt-concentration method has been studied by J. Bonnin, Electricité de France, H. Dumas, Grenoble University, and R. Lievre, Centre National de la Recherche Scientifique. The fluctuations of concentration in the steady state have been observed, and also the spatial distribution when the solution is injected either at the centre or at the wall of the pipe. The effects of brine density and of the wake of the injection nozzle have been studied separately. The method has been used successfully for long conduits (800-1,000 diameters) and appears suitable for lengths of about 200 diameters provided certain limitations are observed.

J. Piquemal, of Laboratoires d'Hydraulique de Toulouse, has developed a method of studying pressure surges in pipelines by an electrical network analogy, and has devised an apparatus which is claimed to give speedy and accurate results.

An electrical analogy is also used in a study of water flow through drowned conduits by W. Jarocki, of Poland, and is concerned mainly with conditions of air entrainment into a closed conduit.

Hydraulic jump is treated in three papers, two of Czech origin and the third Greek. They are "Quantity of Air Drawn into a Conduit by the Hydraulic Jump and its Measurement by Gamma-Radiation" by K. Haindl and V. Sotorník, "Hydraulic Jump in Closed Conduits," by K. Haindl, and "Theoretical and Experimental Analysis of the Hydraulic Jump in a Parabolic Flume," by P. A. Argyropoulos.

Fast water flow in a steep channel is considered by M. Viparelli of Naples University, who points out that in channels of moderate slope the transition from water to air takes place through a geometrical surface, but that in steep channels there is a thick layer consisting of a mixture of air and water, in which the percentage of water decreases from unity to zero. In lower layers, consisting chiefly of water, the law of movement can be represented by Manning's formula, or an equivalent of it. In the transition, unit discharge of water varies according to Gauss's probability equation, the parameters of which are related to velocity and turbulence of underlying water flow.

An analysis for the determination of the rating curve of a venturi channel is presented by Professor A. A. Manzanares, I. S. Técnico, Portugal, who submits a method of theoretical computation taking into account the non-hydrostatic pressure distribution at the critical cross-section. The author also demonstrates that this method of computation gives more accurate results than the classical method which considers the flow lines as parallel in that section. He establishes the general equation of liquid flow in an open channel valid for any type of flow (rectilinear or curvilinear) by introducing a coefficient β, a function of the curvature of the path lines. The relation between the critical depths in the cases of constant flow and of constant energy have been determined, obtaining the theorem of the simultaneity of the maximum flow and of the critical depth. This theorem is then applied to the resolution of the problem of the round-crested spillway. Finally an expression giving the rating curve of the venturi channel has been deduced from the results obtained. Tests on scale models have confirmed the theoretical computations.

Surge Tanks

Surge tanks naturally came in for attention, and it was only fitting that one of the contributors should be Professor L. Escande. In the first part of his paper, the author considers the conditions of stability of a venturi-type surge chamber placed at the commencement of the tailrace of a station which has no supply tunnel and surge chamber upstream of the turbine; in this case only the oscillatory condition is considered. In the second part a more general examination is made of the stability of both normal and venturi-type surge chambers, placed either upstream or downstream of the turbine, and covering the aperiodic as well as the oscillatory condition. The results show that the expression for the surge-chamber section is the same for both aperiodic and oscillatory conditions. In the final section of his paper Professor Escande proposes a device whereby the stability of a tailrace surge chamber is secured much more easily. It consists in making the connection between the tailrace and the surge chamber in such a manner that a pitot-tube effect is obtained, which recovers the dynamic head due to the velocity of flow.

R. Duranton, of Electricité de France, considers the effect of the interconnection of electrical networks and the tendency towards programmed operation on the size of surge chamber. These developments have opened the way for a substantial reduction in surge-chamber sections, and in introducing his paper the author mentioned the possibility of constructing satisfactory chambers to one-tenth of the Thoma condition.

Two papers from Yugoslavia deal with model investigations on surge tanks. G. Hajdin, of Belgrade University, considers the problem of similitude, including the case of distorted models, and K. Djonin, of Avala Hydraulic Laboratory, deals particularly with the problem of distortion in the case of orifice-type chambers.

Flood Disposal

Several papers were contributed dealing, in one way or another, with flood disposal. J. C. I. Dooge, of the Irish Electricity Supply Board, relates that during the course of routine model tests for Carrigadrohid dam it was found that under certain conditions the jets from the ski-jump sluices were unstable. Some of the principal factors discussed are aeration of the sluice gates, the convergence of the sluices, the shape of the bottom edge of the gates,

and the level and position of the upturned toe. The original design of the sluices was modified to ensure that this instability would only occur at reservoir levels well outside the operating range.

Various designs of energy dissipators are described in a paper by E. Levi and G. G. Escamilla of Laboratoire d'Hydraulique de la Secretaria de Recursos Hidraulicos, Mexico. At Solis dam a discharge at velocities reaching 18 m. per sec. was controlled by placing a vertical grid of horizontal steel beams in front of the mouth of the culvert. A somewhat similar solution was adopted at Alvaro Obregon dam, where a scour on the downstream bed was observed, the device in this case consisting of three nests of horizontal cylindrical bars 30 cm. in diameter. In two other cases — Miguel Hidalgo and El Sordo dams — the need to pass debris precluded the use of cross members, and side pillars were used carrying horizontal ribs to guide the water.

A study of particular points of interest arising in model tests on spillways is presented by J. Faure of Chatou Laboratory. These tests relate to a number of Electricité de France and other schemes, and are concerned particularly with the influence of reservoir topography, the spillway profile, the discharge structure, and the stilling bason.

A paper "Kinetic Energy Dissipation on Apron for Gate Dams" was contributed by J. S. Gandolfo and R. D. Cotta of Argentina, in which a study is made of a design of apron downstream of a vertical sluice in which roughly horizontal sections alternate with bucket sections. The effect of the bucket sections is to hold the hydraulic jump within the apron at all discharges, the kinetic energy being dispersed by intermingling with the discharge over the horizontal sections.

Wave Motion

Studies of wave motion as it affects hydraulic as distinct from maritime works are represented by four papers. The paper "The Undular Surge Wave," by O. C. Zienkiewicz and J. A. Sandover, is based on experimental work carried out at Edinburgh University which has enabled a mathematical analysis of surge-wave motion to be built up. This paper is complementary to the article "Experiments on Surge Waves" by these two authors, published elsewhere in this issue.

A mathematical solution of backwater curves in a prismatic or other channel is presented by A. A. Manzanares, of I.S. Técnico, Lisbon.

Seismic waves are considered in a paper by N. N. Ambraseys, of the National Technical University of Athens, and covers the case where the reservoir basin is faulted. This faulting is shown to increase the hydrodynamic pressure, but for ordinary reservoir dimensions the increase is very small. A study is also made of seismic wave heights.

A mathematical study of the reflection, transmission and damping of surges in open channels is contributed by J. Nougaro and A. Barbe of Toulouse.

Some Miscellaneous Subjects

Two interesting papers dealing with practical problems were received from Sweden. H. L. Rundgren, of Vattenbyggnadsbyran, Stockholm, gives an account of laboratory experiments to determine the hydro-dynamic forces acting against semicircular steel cofferdam segments during placing in streaming water. This type of cofferdam, which has been used in Sweden for a number of years for closing temporary openings in dams and for similar purposes, consists of a series of wood-lagged steel arches which are laid horizontally on top of one another until the cofferdam is built up to the required height. The hydrodynamic forces acting on a segment may be resolved into a force acting in the plane of the arch and one acting at right-angles to it. The former is of most significance when the arch is in position, and determines the required strength of the segment; the latter is an uplift force, which is of most significance while the arch is being placed. The investigation was concerned primarily with the characteristics of this uplift force, and as a result a segment cross-section was developed presenting minimum uplift force for any given conditions.

The other paper, by F. Hallhagen and H. T. Kristensen, describes model tests on a log-storage basin in the river Ume just below the Pengforsen rapids. This basin was capable of storing up to 5 million logs, but the decision to build a power station at the rapids made it necessary to determine the effect of the station on the behaviour and capacity of the log-storage basin. The general conclusion was that the necessary storage capacity could be maintained, and operation of the actual station during two log-floating seasons has shown good conformity with model-test results.

The only paper on earth dams came from G. Matta, of Toulouse Laboratory; this is a theoretical study of filtration flow through a vertical dyke.

It is fitting to close this account with the mention of a Portuguese paper by A. Palma Carlos, F. M. Lopes Serra and J. A. Rosado Gusmão, of Direcção Geral dos Serviços Hidráulicos, describing the numerous services rendered by a small site laboratory on the Rio Lis development. Many small hydraulic problems, connected with earth channels, spillways, and so forth, arose which were most conveniently resolved by research facilities on site.

Vancouver Cable Repeat Order

The British Columbia Electric Company Limited have instructed the B.I.C.C. Group to supply and install, at a cost of $3,500,000, an additional circuit in the 138 kV submarine power cable link between Vancouver Island and the mainland of British Columbia. This is to be laid over the same route as the submarine power link completed by B.I.C.C. in 1956, and described at the time in this journal.

The design of the cable will be identical with the earlier one, and will comprise two lengths of the order of 16 miles each and two lengths of approximately three miles each. The two circuits will have a total carrying capacity of some 250 MW, which is believed to be greatly in excess of that of any similar undertaking in the world. As before, the cable will be manufactured at the Trafford Park Works, Manchester, of British Insulated Callender's (Submarine Cables) Limited, and will be installed by British Insulated Callender's Construction Company Limited. The B.I.C.C. Group claims to be the only concern in the world that can manufacture continuous lengths of submarine power cable such as these.

Manufacture of the cable will commence in November 1957, and it is planned that the installation will be complete in July 1958.

Cavitation in Turbomachines

Modern cavitation test facilities are discussed in the light of research objectives. The need is stressed to supplement laboratory measurements by tests on the full scale machine in the power station

By F. A. L. WINTERNITZ, Dipl. Ing. E.T.H., V.D.I.*

PART THREE

Cavitation Test Facilities and Objectives of Research

A comparison of pre-war cavitation test facilities for water turbines and pumps, from a limited survey[56], with recent designs demonstrates the importance which manufacturers attach to cavitation model testing. Since models have become larger and test heads have increased the power installed has gone up substantially. The very considerable expense is accepted by industry as essential to progress. Modern test rigs have many refinements which are not available in the older installations. The Mechanical Engineering Research Laboratory test rigs at East Kilbride may serve to demonstrate this.

Complete model turbines up to 20 in. in runner diameter will be tested in the closed-circuit arrangement in which the water may be circulated by a variable-pitch propeller or a mixed-flow centrifugal pump. A large vertical reabsorber is incorporated to ensure that any air which may have come out of solution in the cavitation zone is not recirculated. Pressure control is automatic, and the air-content control system will maintain the chosen concentration of dissolved air. The rig has a cooling system to keep temperature constant during tests.

Considerable attention has been paid to research requirements which resulted in demands for high accuracy and improved methods of observation. The vertical absorption dynamometers are floating in oil bearings, the torque being balanced by oil pressure and weighed automatically. Turbine speed can be measured with high accuracy by use of a phonic wheel and batch counting. The gauging of flow is by contraction pressure-differential and current meters permanently installed in the tunnel, and head across the turbine is measured by pressure weighing. Continuous remote observation of runner-blade cavitation is possible by use of television cameras, the water being perfectly clear. Some of the initial development work is described in Refs. 57 to 59.

Since this rig was specifically designed for research, there are features in the design of the turbine model and in the instrumentation which will set it apart from similar industrial installations. In the Kaplan model, runner thrust will be balanced by oil pressure and weighed, the friction eliminated by spring supports, and each individual blade hydraulically locked in the test position. Runner-blade and guide-vane setting will be effected by remote control. The overall performance tests may be supplemented by detailed flow investigations to establish the composition of losses.

Among the research objectives scheduled for this rig are investigations into cavitation scale effects, the influence of absolute head, air content and model size, and the influence of relative roughness. For axial-flow turbines the conditions of optimum tip-clearance formation is to be clarified.

The central problem is the development of methods of calculation on which the design of turbine runners with improved cavitation characteristics can eventually be based. Aerodynamic research must here sup-

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REFERENCES TO THE TABLE "SURVEY OF CAVITATION TEST FACILITIES" ON PAGES 430–432

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SURVEY OF CAVITATION TEST FACILITIES

Location	Type O Open ● Closed	Power installed h.p.	Model runner size mm.	Test head range m.	Flow Rates l./s.	Purpose of rig and special rig features	Ref.
AUSTRIA							
Anstalt für Strömungsmaschinen, Graz-Andritz	●	400	Kaplan 300 Francis 200/250 (D. tube)	10—90	200—900	Complete turbine models; flow metered by venturis; head, pressure, torque transmitted by oil pressure and weighed; rig pressure pneumatically regulated.	
CANADA							
Dominion Engineering Works, Montreal	●	300	Francis ~400 (D. tube)	~75—25	~100—1000	Turbine cavitation test rig with resorber and heat exchanger; flow measured by venturis; head, pressure, torque weighed; rig pressure regulated pneumatically.	(1)
FRANCE							
Société Sogreah, Grenoble	●	225	Kaplan 600 Francis 500	0—23	1200	Turbine cavitation test rig with resorber; free surfaces up and downstream of turbine; flow measured by weir; rig pressure regulated pneumatically.	(2)
GERMANY							
Institut für Strömungsmaschinen, Technische Hochschule, Braunschweig	●	20	200	~6	100	Cavitation test rig for axial flow pumps; rig pressure regulated.	(3)
Institut für Strömungslehre und Strömungsmaschinen, Technische Hochschule, Karlsruhe	O	65	350	1—10	250	Cavitation test rig for Kaplan runners; variation of suction pressure by means of two pumps with intakes in turbine tail race.	(4)
Institut für Wasserkraftmaschinen und Pumpen, Technische Hochschule, Stuttgart	● O	260	220	~100	~280	Cavitation test rig for turbines and pumps.	
Kleinschanzlin-Bestenbostel G.m.b.H., Bremen	●	40/100	350	15	600	Cavitation test rig for axial flow pumps.	
J. M. Voith G.m.b.H., Heidenheim-Brenz	O	250/300	Kaplan 257 Francis 300/400	12—15	350—400	Cavitation test rig for water turbines; water of constant temperature supplied by well; non-recirculating; free surfaces up and downstream of turbine; flow measurement by weir.	(5)
GREAT BRITAIN							
Hydraulics Laboratory, The English Electric Co., Rugby	●	182	300	6—17	650	Cavitation test rig for turbines and pumps; flow measured by venturi; pressure control pneumatically regulated ~1·5 to 35 m. w.g. absolute.	
	●	570	300/560	3—275	1700	Cavitation test rig for turbines, pumps and ship propellers with surface resorber; flow measured by venturi; pressure pneumatically regulated between ~1·5 and 56 m. w.g. (abs.).	
Mechanical Engineering Research Laboratory, East Kilbride	●	220/350	250/500	~1·5—70	80—1850	Cavitation test rig for complete models of Kaplan and Francis turbines, with vertical resorber and temperature control; flow measured by contraction pressure-differential and current meters; head, pressure, torque transmitted by oil pressure and weighed; pressure control 2 to 40 m. w.g. absolute; air content control; precision dynamometers supported in oil bearings; remote control of all test conditions; television circuit; circulation pumps with precision speed control.	

SURVEY OF CAVITATION TEST FACILITIES *(continued)*

SURVEY OF CAVITATION TEST FACILITIES *(continued)*

Location	Type O Open ● Closed	Power installed h.p.	Model runner size mm.	Test head range m.	Flow Rates l./s.	Purpose of rig and special rig features	Ref.
Mechanical Engineering Research Laboratory, East Kilbride	●	350	Various	7—180	0—550	Cavitation test rig for centrifugal and mixed-flow pumps with resorber and temperature control; flow measured by venturis; automatic control of pump suction pressure; pressure control ~2 to 27 m. w.g. absolute; air content control; special pulsation and cavitation-free throttle valves; universal mounting for all types of pumps.	
	●	25	~300	—	280	Cavitation test rig for axial flow pump runners; flow circulated by runner on test; measured by contraction pressure differential; precision dynamometer with speed control, mounted on air bearings; pressure control ~3 to 30 m. w.g. (abs.).	
ITALY							
Istituto di Macchine, Università di, Padova	●	75	160/250	12	140	Cavitation test rig for turbines and pumps with resorber.	
	O	~80	150/300	20	500	Cavitation test rig for complete models of turbines and pumps; mounting platform can be raised; maximum suction head 7·5 m.	(6)
Riva Hydraulic Research Laboratory, Milan	●	50	215	15	200	Cavitation test rig for model turbines; free surfaces up and downstream of turbine.	
JAPAN							
Ebara Mfg. Ltd., Tokyo	●	40/60	Kaplan 300/350 Francis 200/350	5—35	165—500	Cavitation test rig for Francis and Kaplan turbine models.	
Fuji Electric Co. Ltd., Kawasaki	O	190	Kaplan 250 Francis 250/300	10—25	350	Cavitation test rig for Francis and Kaplan turbine models.	(7)
Hydraulic Laboratory, Hitachi Ltd., Tokyo	O	170	250/300	35	300	Cavitation test rig for Francis and Kaplan turbines.	(8)
	O	~750	250/300	85	500	Cavitation test rig for Francis and Kaplan turbines.	(9)
Hydrodynamics Laboratory, Kobe Shipyard and Engine Works, Kobe	O	~400	Kaplan 200 Francis 150/250	10—75	100—250	Cavitation test rig for Francis and Kaplan turbine models, with horizontal shaft mounting.	
	O	~230	Kaplan 200 Francis 150/250	15—30	100—250	Cavitation test rig for Francis and Kaplan turbine models, with vertical shaft mounting.	
The Institute of High Speed Mechanics, Sendai	O	120	250	4—11	320	Cavitation test rig for Kaplan turbine models, with variable head race and back pressure control.	(10)
	O	500	300	10—80	340	Cavitation test rig for Francis turbine models.	
Mitsubishi Shipbuilding and Engineering Co., Nagasaki	●	~560	200/300	20—55	300—600	Cavitation test rig for Francis (?) turbine models, suction heads 9·5 m.	
Tokyo Shibaura Denki Co., Ltd., Kamata-Tokyo	O	300	Kaplan 200 Francis 250	53	250	Cavitation test rig for Francis and Kaplan turbines.	(11)

SURVEY OF CAVITATION TEST FACILITIES *(continued)*

Location	Type O Open ● Closed	Power installed h.p.	Model runner size mm.	Test head range m.	Flow Rates l./s.	Purpose of rig and special rig features	Ref.
SWEDEN							
Aktiebolaget K.M.W., Karlstad	●	75	Kaplan 250 Francis 275	12	450	Cavitation test rig for Francis and Kaplan turbines, with horizontal shaft mounting; flow measured by venturi; rig pressure pneumatically controlled.	(12)
SWITZERLAND							
Ateliers de Construction Mécaniques de Vevey S.A., Vevey	●, O	50	200	12	220	Cavitation test rig for Francis and Kaplan turbines, with head across turbine kept constant by float level control (old).	
	O	155	250	42	260	Present test facilities.	(13)
	O	155/310	250/300	42/84	260/520	Extension (new).	
Ateliers de Charmilles S.A., Charmilles	●	100	200	20	300	Cavitation test rig for Francis and Kaplan turbines.	
	●	540	500	25	2000	Cavitation test rig for Francis and Kaplan turbines.	
Escher Wyss Engineering Works Ltd., Zürich	●	800	Kaplan 300 Francis 250/300	100	1100	Cavitation test rig for complete turbine and pump models; flow metered by venturis; head, pressure, torque transmitted by oil pressure and weighed; pressure control suction head +8 to −30 m.; special optical transmissions, for runner observations, bearing friction eliminated.	(14)
	●	365	Kaplan 200 Francis 200	50		Cavitation test rig for Francis and Kaplan model runners.	(15)
	●, O	200	350/400	200		Cavitation test rig for centrifugal pumps.	
Swiss Federal Institut of Technology, Zürich	O	34/64	Kaplan 224 Francis 160 (Inlet diam.)	15/30	300	Cavitation test rig for Francis and Kaplan turbines; flow measurement by venturi, tank or weir; suction heads +8 to −10 m.	(16)
U.S.A.							
Hydrodynamics Laboratory, California Institute of Technology, Pasadena	●	30	~350	—	~110	Cavitation test rig for axial flow pump research: up to three stages; rig pressure ~3 to 20 m. w.g. (abs.); dynamometer with speed control.	(17)
Newport News Shipbuilding and Dry Dock Co., Newport News	O	65	Throat d. tube 250	12	700	Cavitation test rig for turbine models with ~8·5 m. free fall; flow measurement with venturis.	
S. Morgan Smith Co., York	●	300	Throat d. tube ~300	~90	1600	Cavitation test rig for complete Francis and Kaplan models, with temperature control; flow measurement by venturis; dynamometer with speed control and hydraulic load cell take-off.	(18)
YUGOSLAVIA							
Slovenska Akademija Znanosti in Umetnosti, Institut za Turbinske Stroje, Ljubljana-Šentvid	O	~300	D. tube 200/300	25	400	Cavitation test rig for Francis and Kaplan model turbines.	
	●	870	D. tube 350/500	25/50	1000—1500	Cavitation test rig for Francis and Kaplan model turbines.	

The References to this Table can be found on Page 429

plement the verification in the cavitation tests. Although the cavitation testing of model turbines has proved indispensable, it is most desirable that results thus obtained should be verified in actual plant investigations. Published information on such power-station tests is scarce[60]. Yet it is by such measurements only that the problem of scale effect in cavitation can be studied satisfactorily, and it is hoped that the collaboration of plant owners will make truly comparative tests on models and full-scale machines possible.

The research facilities for cavitation tests on centrifugal pumps are generally less elaborate. In the usual open-circuit arrangement the part-load cavitation characteristics are rarely determinable due to the test limitations. These are overcome in a closed circuit pump rig at the Mechanical Engineering Research Laboratory, with most of the features and refinements of the water-turbine test rig. The tendency to increase the load per stage for large units necessitates here too the reduction of $\sigma_{crit.}$ through improved runner design, and a closer study of the effects of conditions at inlet on cavitation and overall pump performance.

In axial-flow pumps the danger of cavitation is increased by comparison with the axial-flow turbine, the problems under investigation, however, being quite similar. In the foreground are researches into the interrelation between number of blades, different hub-to-tip ratios, and efficiency, the decisive factor being cavitation performance. A special closed circuit rig is available for work in this field.

The results of a survey of cavitation test facilities for hydraulic turbomachines in industrial establishments in all parts of the world, as well as at centres of research and technical universities, are tabulated in a form suitable for reference and comparison. The author would be pleased to receive information about any other cavitation test facilities.

Conclusions

Present-day trends in the design of hydraulic turbomachines have enhanced the need for systematic cavitation research and adequate research facilities.

The physical aspects of the phenomenon and the factors controlling it are as yet not fully understood. Experimental evidence indicates that a definite cavitation pressure does not exist, and that scale effects may occur. Their practical importance in the cavitation testing of hydraulic turbomachines awaits assessment.

Although cavitation attack is generally regarded as essentially a fatiguing process, and therefore a mechanical form of material destruction, there is no consistent definition of the location of the zones of damage in relation to the phases of the cavitation cycle.

Cavitation parameters used with turbomachines are essentially linked to machine performance and do not relate to the probability of damage. Improvement in the cavitation performance must extend to the damage aspect.

In designing for optimum cavitation characteristics, the turbomachine must be treated as an integral unit of components contributing to the ultimate performance. Contour and roughness separation should be eliminated by correct passage contour and surface finish. Pressure distribution in the runner must be without excessive suction peaks, which calls for the development of suitable blade sections and correct leading-edge formation.

The cavitation model test has proved an indispensable tool. Under investigation are the possibilities of scale effects, model size, absolute test head, air content and physical properties of water. Complete similitude between model and full-scale case cannot be achieved.

Modern test facilities are designed for investigations under actual heads and on models large enough in size to ensure operation outside the transition range. Close control over the parameters of the test is possible, and accuracy of measurement is high.

Model investigations should be supplemented by measurements on the full-scale machine in the power station in a final corroboration of results. From the available evidence it would appear that very low pressures are required to trigger off cavitation when relatively large nuclei are absent, and that inception at vapour pressure indicates their presence.

Acknowledgment

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"A Guide to Valve Practice." Sir W. H. Bailey & Co. Ltd., Manchester, are celebrating their 125th Anniversary, and in commemoration of the event they have issued an attractive 64 page stiff-covered booklet under the above title. The booklet tells the story of the firm from its foundation, and includes a section of quaint illustrations depicting quainter products, such as a hydraulic organ blower, a beer raiser, a bell chiming machine, a "mysterious clock with automatical hands," and an exhaust-operated motor horn "that will not irritate the most sensitive person." The main part of the booklet, however, consists of an illustrated summarised catalogue of the wide range of valves, boiler fittings, etc., made by this firm. In the hydraulic section sluice valves are shown up to 54 in. in diameter.

George Fischer Limited. This Swiss firm, which has been despatching high-quality +GF+ steel castings to hydro-electric plants in all parts of the world during the last 50 years, has published a handsomely illustrated brochure, in English, to show the up-to-date techniques used in their works, especially their non-destructive methods of testing.

Book Reviews

Colorado-Big Thompson Project. Vol. I. Published by the United States Department of the Interior, Bureau of Reclamation, and obtainable from The Superintendent of Documents, U.S. Government Printing Office, Washington 25, D.C., 1957, 10½ in. x 8 in. x ⅝ in., 187 pp., 118 ff. Price $2.75.

The chief objective of this project was to assure the water supply to some 600,000 acres of irrigated land in north-west Colorado, with incidental water-power development as a secondary but important supplement. Total power developed, excluding the 8,500 kW generating capacity of the Flatiron pumping unit, amounts to 175,450 kW, and in planning the works opportunity has been taken to extend the natural recreational facilities of the region in conjunction with the National Park and Fish and Wildlife Services.

The volume gives a technical account of the entire project in regard to planning, legislation, the works constructed and the geology of the area. Further volumes will discuss respectively: dams and reservoirs (Vol. II); the major waterways (Vol. III); and the power and pumping-plant stations which have been erected (Vol. IV).

In compiling the volumes, the objective has been to make selective use of the vast amount of data that have been accumulated so as to give full emphasis to unusual and technically interesting features while devoting a minimum of space to conventional practice and well-established procedures. Consequently the text follows a pattern that makes it of the greatest use to engineers who are engaged on similar or allied work. It is admirably thorough and illustrated at all points.

Talsperren-Barrages-Dighe (Dams), a special issue of *Wasser- und Energiewirtschaft — Cours d'Eau et Energie*, Vol. 48, No. 7-9, July-August-September 1956, published by Guggenbuhl & Huber Verlag, Hirschengraben 20, Zurich 1; 11¼ in. by 8 in., 136 pp., 170 ff. Price in Switzerland, paper covers, Sw. Fr. 9; bound in boards, Sw. Fr. 12.

A handsome and highly interesting publication issued on the occasion of the 45th yearly meeting of the Swiss Water Association held in Flims-Waldhaus on 30th and 31st August 1956, and devoted exclusively to dams. Descriptions are restricted to a minimum, and stress is laid on outlining the multiple problems encountered in dam building. For this and for the selection of an outstanding team of contributors, special credit must be given to the Editor, Dipl.-Ing. G. A. Töndry. We regret that lack of space compels us to restrict this review to a list of contents. Except where otherwise indicated, the articles are published in German. "The Zervreila Dam and its Power Station," by Dipl. Ing. Michael Kohn, Motor-Columbus A.G., Baden; "Geological Assessment of Dam Sites and Storage Reservoirs," by Prof. Dr. Rudolf Staub, ETH Zurich (Federal Technical University); "Theories Relating to the Calculation of Concrete and 'Fill' Dams," by Professor Gerold Schnitter, ETH; "Tests on Dam Models" (in French), by Prof. Dr. Ing. Guido Oberti, Director of ISMES Laboratory, Bergamo; "Gravity Dams," by Henri Juillard, Bern; "Light Gravity Dams" (in Italian), by Claudio Marcello, Director of Società Edison, Milan; "Arch and Dome Dams" (in Italian), by Dr. Ing. Carlo Semenza, Director of the SADE concern, Venice; "Dams and their Ancillary Works" (in French), by André Coyne; "Arch Dams in Switzerland" (in French), by Alfred Stucky, professor at Lausanne Technical University; "Selecting the Profile of Earth and Rockfill Dams," by Charles Schaerer, head of Earthworks Division at EHT Laboratory; "Problems of Flood-Control Works," by Dipl. Ing. Conrad Schum; "Dam Concrete" (in French), by Dipl. Ing. Olivier Rambert, Electro-Watt, Zurich; "Borings and Grouting in Connection with Dams," by Dipl. Ing. Charles E. Blatter, Director of Swissboring Co., Zurich; "Installations of Dam Construction Sites," by Dipl. Ing. Adolf Schlaepfer, Zurich; "Applying Heavy Contractor-Equipment to Dam Building," by Dipl. Ing. Max Oswald; "Behaviour of the Rossens Dam during its First Years of Service" (in French), by Henri Gicot, consulting engineer; "New Methods for the Geodetic Determination of Deformations in Dams and other Constructive Works" (in French), by Professor Dr. Walter K. Bachmann, Lausanne T.U.; "The Fifth International Large-Dam Conference, Paris, 1955," by Harald Link, Frankfort o/M. The impressive bibliography appended to the volume has been compiled by M. Kohn, H. Link, N. Schnitter and C. Semenza, and includes no fewer than 300 items.

The Conservation of Natural Resources. The Institution of Civil Engineers, Great George Street, Westminster, London, S.W.1. Price to members 7s. 6d.; to non-members 17s. 6d.

This booklet contains the text of seven lectures recently delivered at the Institution by men who have made a life study of this particular subject, and who view with alarm the rapid expenditure of resources such as metals, land, and fresh water, which increasing industrialisation is bringing about.

Sir Harold Hartley, G.C.V.O., in a foreword summarises the findings, pointing out how the lectures were the sequel to the United Nations Conference on Conservation at Lake Success in 1949, the Paley Report of 1952, and the work of the Conservation Foundation of New York. Sir Harold Hartley's 1955 Graham Clark lecture on "The Engineer's Contribution to the Conservation of Natural Resources" forms the first of the lectures, and a further lecture of particular interest to our readers is that by Mr. G. M. Binnie on "The Control, Storage, and Use of Water." Other lectures deal with minerals, land, air pollution, metallic corrosion, and the use of waste materials.

British Standards Annual Report, 1956-7. British Standards Institution, 2 Park Street, London, W.1. 270 pp. Price 7s. 6d.

This report gives some idea of the immense range of work of the British Standards Institution, for it serves about 60 major industries. Over 3,000 British Standards are now current, and during the year under review the sale of copies exceeded the million mark for the first time. The Institution has continued its collaboration in international activities, and a section of the volume is devoted to progress reports from the International Electrotechnical Commission and other bodies.

Protecting the Prestressing Rods at Allt-na-Lairige Dam

Fig. 1. Bar-wrapping shed, showing wrapping machine in use

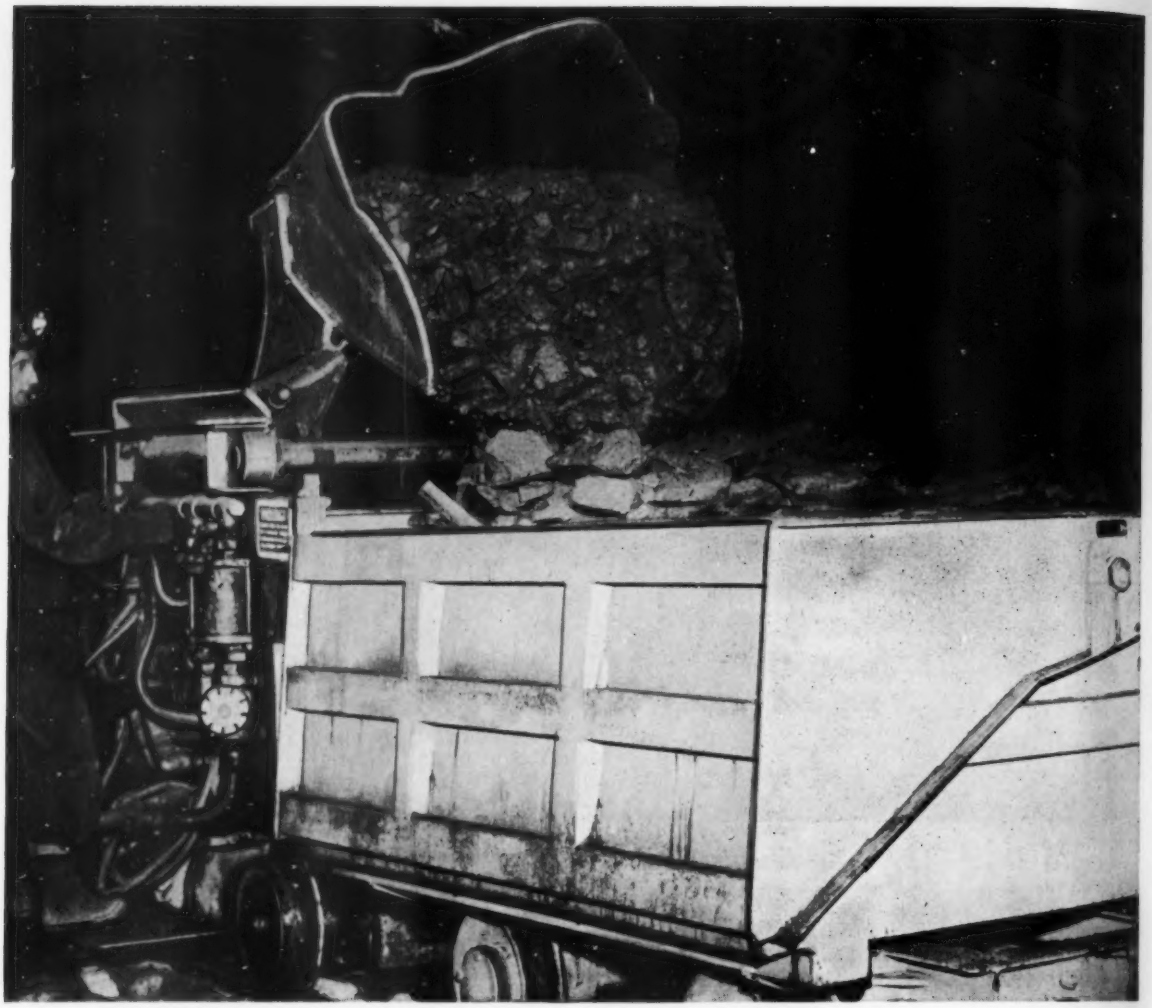
THE North of Scotland Hydro-Electric Board's Allt-na-Lairige dam (illustrated in WATER POWER July 1955, p. 262) is the first large dam to be built in Western Europe in which prestressed concrete was used employing the Lee McCall system of post-tensioning. The advantages of this method and the techniques involved do not come within the scope of this article, although it can be stated briefly that prestressing represents a considerable saving of concrete. The basic idea is to anchor the dam to the foundation rock by means of groups of high-tensile steel rods, passing through the concrete wall of the dam and tensioned by hydraulic jacks against steel top anchor plates. Thus a much lighter form of construction is possible, since the forces required to stabilise the dam, obtained by dead weight of concrete in a plain gravity structure, are mobilised by the tensioning of the rods. Incidentally, as a result of this economy the Board decided to build the dam higher, thus increasing the potential power and storage capacity of the reservoir.

Of the many problems which presented themselves to the engineers in the initial stages, not the least was a way of preventing the bonding of the steel rods to the surrounding concrete as it was placed; for this would have obstructed their final tensioning. Several tests were carried out with a view to finding the most suitable means of isolating the rods from the concrete in such a manner that freedom of movement to the rods within the concrete during tensioning was assured. The most satisfactory way of accomplishing this proved to be by coating the rods with a grease-type wrapping. It was decided, therefore, to give Denso treatment to the rods which were to be wrapped—those at the bottom of the anchorage pits, the second lift bars, the top bars and certain of the lower top bars. This in turn not only solved the problem of allowing movement for the rods, but at the same time ensured their freedom from corrosion. The methods of application were those usually followed and may usefully be described:—

The rods, of high-tensile steel $1\frac{1}{8}$ in. in diameter, were delivered from the manufacturers in Sheffield direct to site in sections, having been precoated with a protective varnish as a short-term anti-corrosion measure. Deliveries were regulated to make certain the rods were not stored on site for longer than was necessary, and they were housed and treated in a special wrapping shed, a corner of which we illustrate.

Before wrapping, rods were stripped of their protective varnish and thoroughly primed with Denso paste—the normal Denso undercoating.

The rods were then wrapped with standard Denso tape and an overwrapping of Denselt tape, both tapes being applied simultaneously by a special wrapping machine (as may be clearly seen in the illustration). Denso tape is based on saturated petroleum hydrocarbons, and, when wrapped over metal, forms a waterproof plastic sheath which, it is claimed, never hardens cracks nor sets. On the other hand, Denselt



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tape is a tough, fully impregnated and thickly coated bituminous wrapping which gives added mechanical support to the soft grease-like coating of Denso tape underneath. Rods thus wrapped to the end threads were positioned and joined one to another by screwed couplings.

These couplings have themselves been the subject of careful tests. It was evident that the enlargement in diameter occasioned by the coupling would create added resistance during tensioning, and here again the normal Denso procedure proved satisfactory. On either side of each coupling where it connected the rods, a smooth surface was built up by the application of Denso Plast, so that the joint could be wrapped to form a continuous even contour from rod to rod. By this means all danger of the jointed rods binding with the concrete during tensioning was eliminated.

As the foundation concrete of the dam was raised to rock surface level, temporary shafts were extended to a given height, and steel staging erected over the shafts. Cages, containing the upper and top rods, which had been assembled on the ground, were then hoisted by derrick crane and lowered into position in the temporary shafts, the rods being coupled in the manner already described. After final tensioning—some of the jointed rods stretching to a maximum of 5 in., the average being 3½ in.—the top anchorages and temporary shafts were concreted in.

The consulting engineers for the Allt-na-Lairige project were Babtie, Shaw and Morton, and the manufacturers of the Denso products used in the scheme were Winn and Coales Ltd., Chapel Road, London, S.E. 27.

Expansion-Joint Sealants

A range of expansion-joint sealants, developed by Products Research Company, Los Angeles, California, and widely accepted in the United States, is now being made under licence in Great Britain by British Paints Limited, Curzon Street, London, W.1. They are synthetic-rubber compounds which are claimed to have properties of adhesion, flexibility and durability never previously obtainable, thus being particularly suitable for hydraulic structures.

A wide variety of these P.R.C. compounds, as they are termed, is offered, each one specially formulated for a particular job, but they generally form two distinct groups. Those in the first group, normally known as two-part materials, are supplied in fluid form in two separate containers consisting of a base and an accelerator. They are mixed together immediately before use and cure chemically at atmospheric temperature, *in situ*, to strongly adherent high-performance rubbers. The cure is independent of atmospheric oxygen, and in fact, the materials cure faster when completely submerged in water than they do in air at the same temperature. Sealants in this group are available in two consistencies: the "flow" type, which is pourable and brushable and is designed for horizontal application or where risers are used; and the "heavy" type, which is a non-sagging compound designed for application by caulking gun, spatula or putty knife to vertical or overhead surfaces.

Those in the second group, known as one-part materials, are supplied in liquid or putty consistency

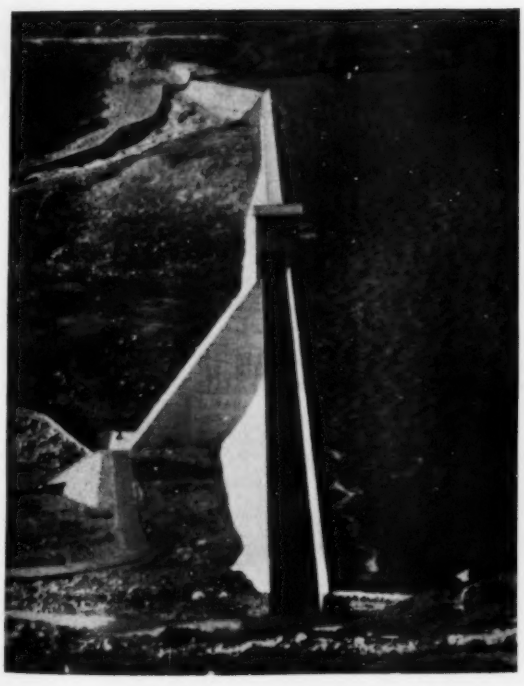
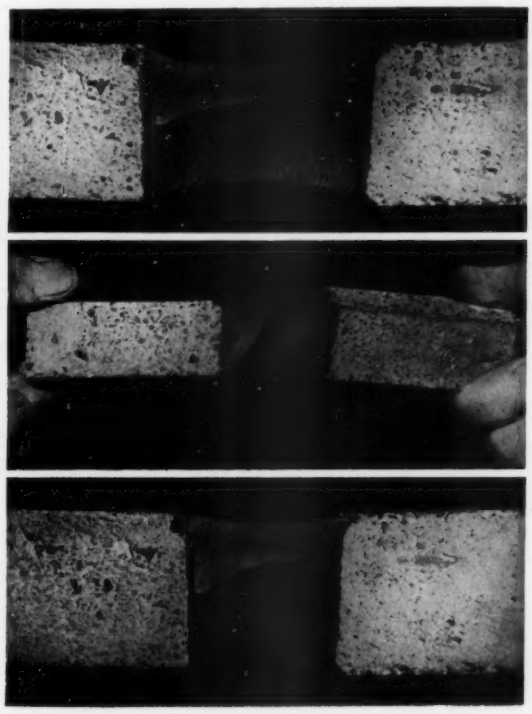
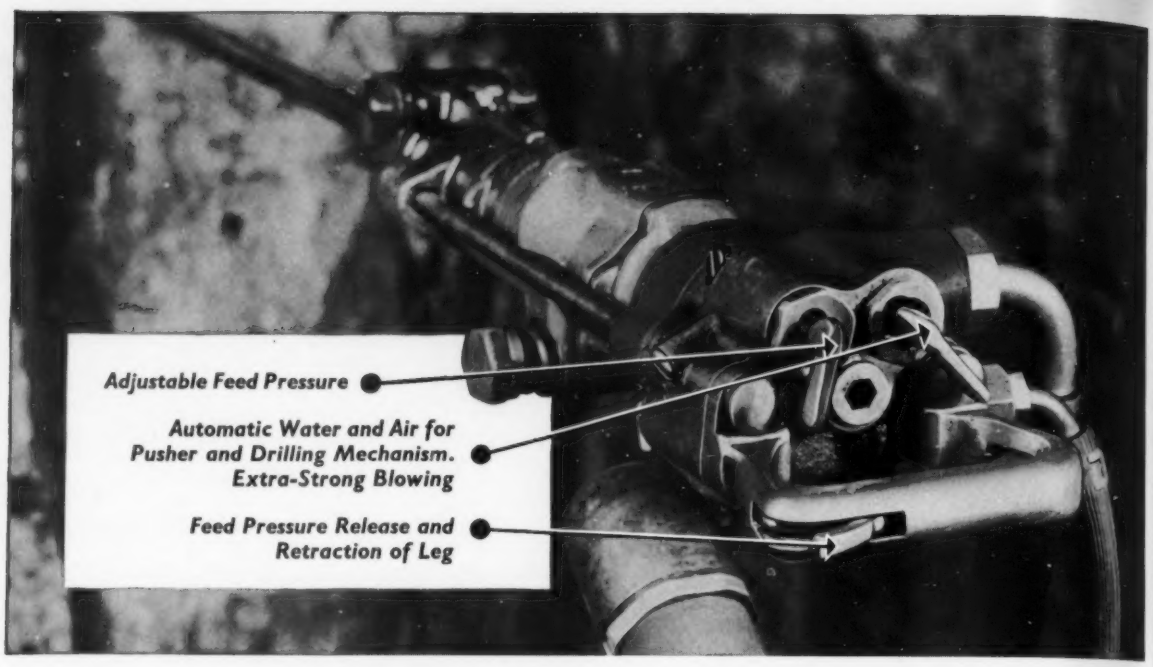


Fig. 2. Allt-na-Lairige dam. Note the slender proportions made possible by prestressing



P.R.C. sealant test specimen stretched, twisted and released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Automatic Water and Air for Pusher and Drilling Mechanism. Extra-Strong Blowing
Feed Pressure Release and Retraction of Leg

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Retractable leg saves time

When the leg has to be moved the feed pressure is easily released by squeezing the hand grip. By further pressure on the grip the leg retracts automatically. When the leg is in the new position suitable for continuous drilling, retraction stops and the feed pressure comes back by loosening the grip of the hand. *All this can be done while the drill is still running.*

This new idea of a retractable leg enables quicker repositioning of the leg and reduces the number of steel changes, thereby increasing footage per manshift. When drilling high holes it is now far easier to alter the position of the leg more frequently in order to maintain an optimal feed angle and feed pressure.

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The Lion has a drilling rate at least 30% higher than other rock drills of the same weight. Furthermore, it can maintain its high speed *even when drilling deep holes.* This means quickly drilled deep hole rounds and a faster advance. The Lion also reduces *to a minimum* the gauge wear of the bits in abrasive rock. And owing to the ease with which the feed pressure is released and brought back into action, the Lion is a *handier* drill to work with in fissured rock.

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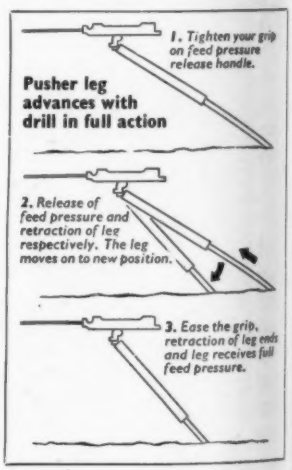
1. Tighten your grip on feed pressure release handle.
Pusher leg advances with drill in full action
2. Release of feed pressure and retraction of leg respectively. The leg moves on to new position.
3. Ease the grip, retraction of leg ends and leg receives full feed pressure.

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and are ready prepared for use. After application they convert to high-performance rubbers by evaporation of the solvent.

In all cases the concrete surfaces are prepared by a primer to control water seepage and produce maximum adhesion. Two types of primer are available, one or other of which is recommended according to the conditions to be met.

The illustrations shown on p. 436 depict a test specimen stretched, twisted, and finally released. These illustrations relate to a sealant seam $\frac{1}{2}$ in. square by 2 in. long bonded between two concrete blocks. Tests on a number of these specimens of both flow and heavy types have shown the adhesive strength in tension to be 120 lb. per sq. in., and the extensibility after curing to be 200% and after prolonged water immersion 150%. The sealant is stated to remain flexible at $-40°$F. and to be proof against softening or flowing at 225°F.

Chief Joseph Runner

With shipping preparations almost complete, the second runner built by the S. Morgan Smith Company for the Chief Joseph dam is shown below before it left the Company's York, Pennsylvania, plant for the dam site on the Columbia River near Bridgeport, Washington. This 100,000 horsepower hydraulic turbine will be the 16th unit installed at this hydro-electric station, which will ultimately contain 27. The first of the six turbines ordered from the S. Morgan Smith Company in 1953 (unit 15) went into commercial operation at the dam early in August. Unit 16, which is shown below on the new S. Morgan Smith special railroad wellcar for turbine runners, will, like the other units, operate at 100 r.p.m. under a head of 165 ft. in this U.S. Army Corps of Engineers project.

British-Made 125 h.p. Tractor

International Harvester Company of Great Britain Limited have announced plans to build at their Doncaster works a British-made version of the 125 h.p. tractor which will weigh 13 tons without equipment. The new TD-18 machine is the second largest made by the Company. Its performance ranges from working a 12 to 16 cu. yard scraper, operating a 3 cu. yard shovel with a break-out force of 27,000 lb., to winching loads with a maximum pull of nearly 20 tons at the speed of 77 ft. per min. It is considered the best size for heavy earthmoving jobs. These varied applications mean special matching equipment and it is understood that steps are being taken to make appropriate bullgraders, scrapers, cable control units, winches and pipebooms available very shortly.

The new tractor is provided with an improved track-frame construction and simplified controls include enhanced visibility and power-assisted steering. Unit-type construction is claimed to make for easy service and to simplify maintenance work. The power unit is a 6-cylinder Rolls-Royce diesel engine delivering 125 h.p. at 1,460 r.p.m.

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Herakles the Archer, *by Antoin Bourdelle*
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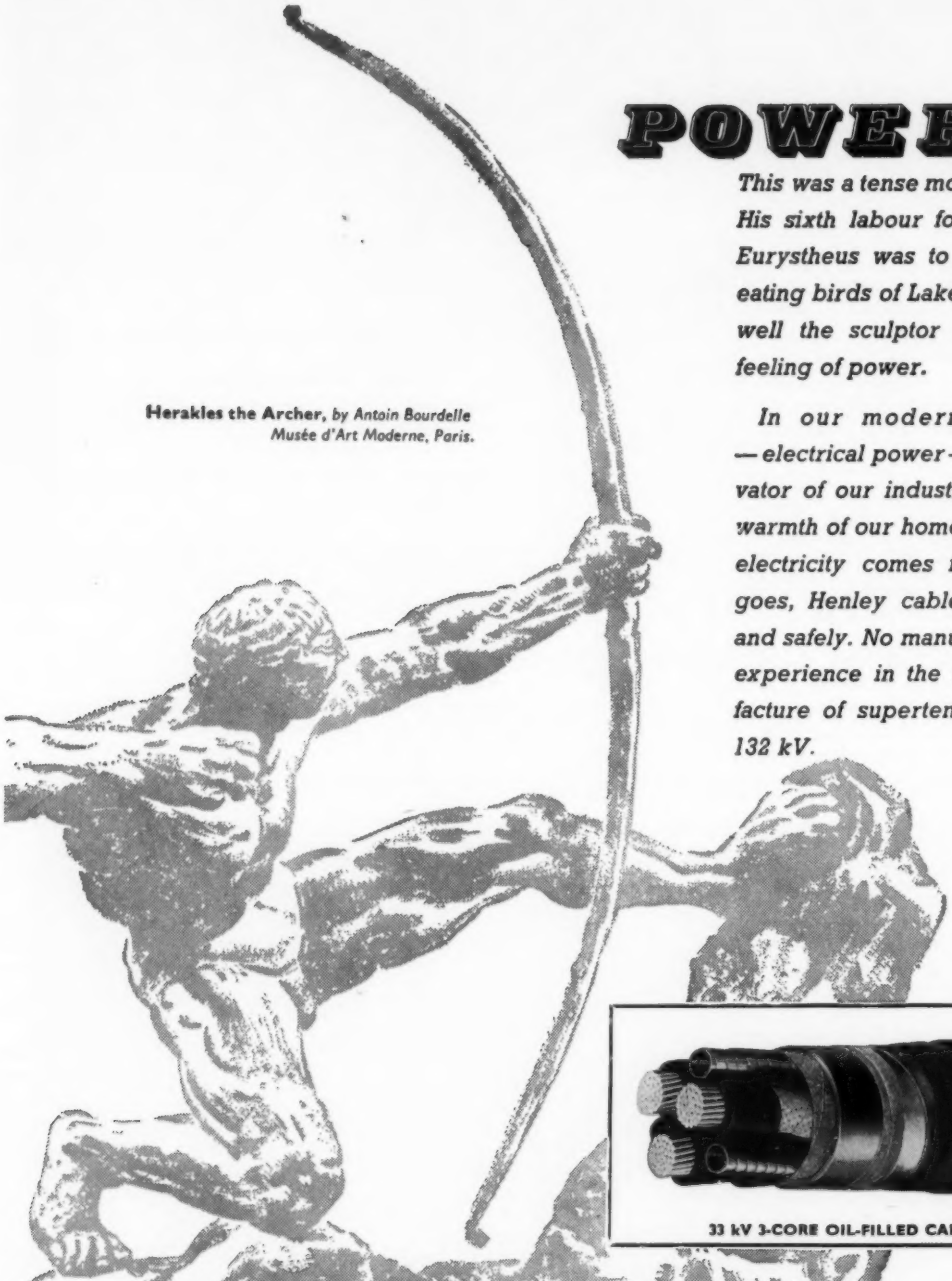

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Abstracts from the World Technical Press

Colorado River Project

The contract, Glen Canyon dam, the key to the entire Colorado River development, was awarded on April 29 to Merritt-Chapman and Scott, of New York, whose bid of slightly under $108 million was not only the lowest submitted but also proved to be more than $27 million below the estimate of the project construction engineer. Preliminary work on the dam site began as early as October 15, 1956, and work on the dam itself is now in progress. Merritt-Chapman and Scott's contract calls for the following major items: (1) Drilling of left diversion tunnel (the right diversion tunnel was included in the preliminary work and is scheduled for completion for December, 1957); (2) Lining both left and right diversion tunnels; (3) Building two cofferdams to divert the river during construction; (4) Constructing the concrete dam from bedrock to crest; (5) Construction of the power house and related features, and (6) Drilling and lining the spillway tunnels. The total volume of concrete to be placed will be 5·2 million cu. yards, of which 4·77 million will be for the dam proper. Time allowed for completion of the prime contract is 2,500 days, i.e. almost 7 years. Once completed, Glen Canyon dam will be the third in the world as to height and reservoir size, and the seventh largest hydro-electric power plant in U.S.A. (C. H. Carter, Assistant Regional Director, Bureau of Reclamation, *The Reclamation Era*, Vol. 43, No. 3, August 1957, p. 53, 6 pp., 9 ff.)

Penstock Coating Rig

The special rig devised by Gustafson Coating and Blasting Co., of Coeur d'Alene, Idaho, for sand blasting and coating the inside of the 16 penstocks now in operation at Chief Joseph Dam hydro-plant is described briefly. This rig is a rubber-tyred movable staging assembly, riding on a slope of 45°, and fitted with adjustable steel jaws for positioning and levelling the work platform. The platform proper, roughly elliptical in shape, was 32 ft. in length and big enough to fill the penstock cross-section and make it impossible for the workmen to fall. The procedure was as follows: Sand-blasting of the steel was the first step, followed at once—before the sand-blasted surface could oxidise—by a primer coat of cold coal-tar enamel. Final step was the application of the hot coal-tar enamel. Heating of the enamel was done right in the penstocks, at a point some 100 ft. from their top, in heating kettles holding each 1,000 lb. of coal-tar which took about 2 hours to reach 500°. The heated material was delivered in insulated buckets up or down the penstocks on a cable-drawn buggy to the work platform. Fans pulled smoke and fumes from the penstocks as coating progressed. All workmen wore respirators, and those attending to coating proper (or, as it is called in the original article: "dobbing") wore out three or more 6 in. brushes each eight-hour shift, as the $\frac{1}{16}$ in. protective coat was applied at the rate of 1 lb. per sq. ft. Once the penstock was coated, the entire inside surface was submitted to the spark-test of a 10,000 V electric detector, and any holes or flaws found were rectified at once. A crew of eight men took four days to coat one of these big penstocks, 25 ft. in diameter and 305 ft. in length. Each setting-up operation inside the pipe and each dismantling of the rig for moving to the next penstock took one additional day, which means that about one-third of the total working time was spent in rigging and preparatory jobs. To save time, sand-blasting could proceed in one penstock while hot-coating was being applied in another. The contractors claim that their protective coating should last 50 to 100 years, and withstand temperatures from −4 to 120°F. (*Engineering News-Record*, Vol. 155, No. 20, pp. 45-46, 3 ff.)

Cavitation and Water Tunnels

As a hydraulic analogue of the aerodynamist's wind tunnel, the water tunnel has now become the hydraulic engineer's most powerful tool for the investigation of the phenomena of cavitation and of its effects on hydraulic machines and structures. In addition to an outline of cavitation itself and its effects, this paper, presented at the 26th Annual Research Committee Meeting held July 1956 in Colombo, gives a general description of the water tunnel, and of its hydraulic and mechanical features, and discusses some of the types of water tunnel now in use in leading research laboratories. (K. Seetharamiah, *Irrigation and Power*, Vol. 14, No. 1, 1957, p. 66, 22 pp., 19 ff.)

The Voltabarozzo Experimental Station

A detailed description is given of this important experimental station, set up by the Institute of Hydraulics and Hydraulic Structures of the Padua University. On the whole, the station consists of a large-scale model (1 : 25) of the works now in course of construction for the routing of Adige floods into the Lago di Garda. The model alone covers an area of 150 by 120 m., and is designed for a discharge of 650 litres per sec., which corresponds to Adige floods of 2,000 cu. m. per sec. (Aldo Rossi—Francesco Marzolo *Giornale del Genio Civilo*, Vol. 95, No. 3, March 1957, p. 155, 8 pp., 8 ff., 3 folding plates.)

Cavilla Power Station

A detailed description is given of this plant, which together with the Arsié station, farther upstream, constitutes the initial stage of the Cismon Torrent development. A double-curvature 70 m. high dam near Corlo creates a reservoir 45 million cu. m. in capacity, which collects the discharge from the Arsié plant and supplies Cavilla with power water through a diversion tunnel 3·8 m. in diameter and 3,100 m. in length. The tunnel, which houses the penstock serving the two generating sets of the underground machine hall, and the two-chamber surge tank, have been excavated in calcareous rock and are lined with segments of reinforced cement. The useful average discharge is 22 cu. m. per sec., maximum discharge being 40 cu. m. The vertical generating sets consist each of a 17,500 kVA alternator driven by a Francis turbine at a rated speed of 333·3 r.p.m., run-away speed being 640 r.p.m. The open-air substation is equipped with a 24,000 kVA and a 35,000 kVA transformer, the former feeding the 130 kV system and the latter, the 220 kV network. The generating sets are controlled either auto-

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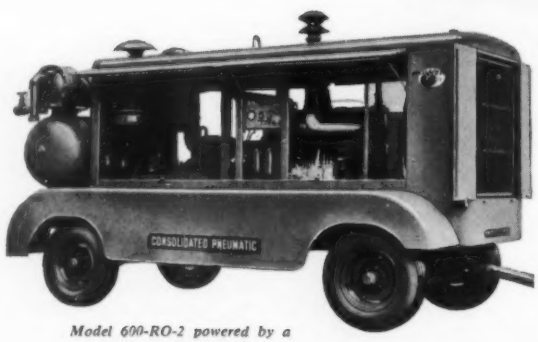

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matically or by hand from the control room. Most of the electrical equipment, inclusive of the two alternators and the 35,000 kVA transformer, were manufactured and erected by Tecnomasio Italiano Brown Boveri, and the turbines were supplied by Theodore Bell & Co., Kriens-Lucern, Switzerland. (*Rassegna Technica Tecnomasio Italiano Brown Boveri*, Vol. XVIII, 1957, p. 3, 14 pp., 19 ff.)

Fluid Mechanics and Heat Transmission

An account is given of the meeting held on November 15, 1956, by the Technical Committee of the Société Hydrotechnique de France with the co-operation of A.T.E.N., the French Association for the Production and Utilisation of Nuclear Energy. In fact, we have to deal here with a symposium of heat transfer considered in its relationship with fluid mechanics. Among the nine papers read at the meeting, we should like to single out "Laminar and Turbulent Convection" by Professor G. Ribaud, and "Current Notions on the Mechanics of Turbulence and Turbulent Heat Transfer" by A. Craya, Grenoble University. (*Mémoires et Travaux de la Société Hydrotechnique de France*, Supplement to Vol. 1, 1957, and *La Houille Blanche*, Vol. XII, No. 1, January-February 1957, 100 and 123 pp. respectively, 90 ff.)

Earth Dam Studies

Two research engineers of the Secretariat of Mexico's Hydraulic Resources discuss in this paper, from the aspect of soil mechanics, some of the problems confronting the designers and builders of earth dams. Construction materials, laboratory tests and classification of representative samples, standard compaction, and shearing strength, are dealt with in separate paragraphs. There are also sections on cohesive and intermediate soils, as well as on rock fills. (Raul J. Marsal and Enrique Tamez, *Indian Journal of Power and River Valley Development*, Vol. VII, No. 3, March 1957, p. 9, 7 pp., 6 plates, 5 ff.)

Tunnelling Machines

The purpose of this note is to review recent developments in machines intended for tunnelling into harder rock, such as encountered in mining and hydroelectric development. Following an outline of early and later developments in America and Europe, and of the practical experience gained in mining operations, a general analysis of the problems arising in tunnelling is given. A description of various types of tunnelling machines appears in two appendices. (G. S. Marwaha, *Indian Journal of Power and River Valley Development*, Vol. VII, No. 1, March 1957, p. 15, 6 pp., 5 ff.)

Controlling Sand Moisture in Dam Concreting Plants

The main difficulty encountered in the accurate proportioning of batching water—one of the most important prerequisites to the manufacture of concrete of uniform quality—lies in the fluctuation of the water content of the aggregating materials, a fluctuation which occurs in its highest degree in sand. Two methods have been heretofore applied to the analysis of sand moisture, one in which a determined quantity of sand is weighed, dried and reweighed, and the other in which the gas pressure produced when mixing a small quantity of wet sand with carbide powder in a pressureproof bottle is ascertained. These two methods unfortunately have the disadvantage of requiring some minutes before the water content of the sand is ascertained, and this disadvantage cannot be underestimated with highly efficient batching towers capable of turning out 3 to 6 cu. m. of concrete per minute. Confronted in 1955 at the concreting plant of the Mauvoisin dam by stress requirements substantially exceeding those previously accepted for concretes to be used in high dams, the author worked out a new method which he describes in this article. To put it briefly, the method rests on the measuring of the electrical resistance of wet sand, and the equipment involved consists essentially of a sand-weighing steel tank in which a fixed cross-shaped probe of sheet steel is attached to a suspension rod so as to hang at about one third of the height of the tank. Both probe and suspension rod are rubber-insulated against the wall of the tank. The resistance between probe and the wall of the tank is measured by means of a calibrated switchboard-ohmmeter fitted on the control desk and giving direct percentage readings of the water content of the sand in full view of the attendant. The ohmmeter is fed from a dry battery. It is claimed that this device, which was contrived on the basis of numerous experimental tests, gives very good results, especially when fluctuations of the water content occur in quick succession. When required, it is perfectly possible to adjust each individual mix before transferring it to the mixer. For example, a sudden fluctuation of 7% to 13% can immediately and smoothly be counteracted. (P. Graf, S. A. Conrad Zschokke, Geneva, *Schweizerische Bauzeitung*, Vol. 75, No. 6, February 9, 1957, pp. 85/86, 3 ff.)

Precipitation over Southern Ontario

An account of the investigations undertaken with a view to obtaining preliminary estimates of probable maximum precipitation over Southern Ontario. The subject of this paper, which is reproduced by permission of the director, Meteorological Branch, Department of Transport, Ontario, is of particular interest at present to hydraulic engineers, in view of recent unprecedented storms such as Hurricane "Hazel." (J. P. Bruce, Department of Planning and Development, Ontario, *The Engineering Journal*, Vol. 40, No. 7, July 1957, p. 978, 7 pp., 3 ff.)

Note. The above mentioned journal also contains a report on the progress of work on the St. Lawrence Seaway and Power Project (p. 1,003, 3 pp., 2 ff.).

Head Losses in Cylindrical Pipes

This problem can today be considered as solved on sound well-proven theoretical bases, as far as it relates to cylindrical pipes of circular cross-section. The author of this paper, proceeding from experimental tests, extends the problem to cylindrical pipes of non-circular section, the walls of which also present a varying degree of roughness, and claims it is possible to show: (1) that, in general, the friction velocity varies little around the perimeter of any cylindrical section of a pipe, the wall of which does not vary in roughness; (2) that the head-loss formula applied to any cylindrical pipe of homogeneous or heterogeneous roughness is identical to the formula applied to pipes

of circular section, subject to introducing a "mean relative roughness factor," which it is possible to determine graphically, and a "shape factor," the magnitude of which can be asesssed. (Professor A. Fortier, Sorbonne, *La Houille Blanche*, Vol. XII, No. A, May-June 1957, p. 325, 7 pp., 3 ff.)

Development and Problems of Energetics in Yugoslavia

This is an English translation of an article written on the occasion of the World Power Conference's 11th Sectional Meeting, held in Belgrade, 1957, it is a detailed survey of power development in Yugoslavia, including the energy balance sheets for 1952 and 1956, and a survey of hydro-electric developments at present in operation or in course of construction, as well as new projects. *(Vladimir, Slebinger, Elektrotechniski Vestnik*, Vol. XXV, March-April 1957, p. 7, 9 pp., 12 graphs, 1 map.)

The Umtru Project, Assam

A detailed description is given of this small but first major hydro-electric project in Assam, on which construction work has just been completed. At the head of the Umtru Rapids, the river water is impounded by a masonry diversion dam and diverted through a 4,600 ft. long conduit system to the power house located two miles farther downstream. This conduit system consists of two tunnels 2,700 and 830 ft. long, connected by a steel pipeline of 705 ft. A 350 ft. long high-pressure steel penstock connects the second tunnel to the power house. The penstock branches into three distributor pipes for feeding the three turbines. A surge tank 32 ft. in diameter and nearly 100 ft. high above the tunnel is inserted in the middle of the second tunnel. The turbines are Francis 3,500 h.p. vertical units operating under 175 ft. of effective head and running at 500 r.p.m. The generators are twelve-pole, 3,600 kVA units, 330 V, 50 cycles. The Umtru project was carried into effect with the aid of the Colombo Plan and the technical co-operation of the Canadian Government. (G. C. Goswami, Chief Electrical Engineer, Assam, *Indian Journal of Power River Valley Development*, Vol. VII, No. 7, May 1957, p. 2, 6 pp., 11 ff.)

Corps of Engineers' Research for Better and Cheaper Concrete

The Corps of Engineers, as one of the largest users of Portland cement in the U.S.—4 million barrels a year in civil work alone—naturally encounters many problems in its use. To search out for solutions which will result in better or cheaper concrete, the Corps maintains extensive facilities for concrete research at the Jackson, Missouri, sub-office of its Vicksburg Waterways Experiment Station. There, a million-dollar plant of the concrete division includes petrographic, X-ray, chemical, thermal, physical and concrete laboratories, plus a 2 cu. yd. semi-automatic batching and mixing plant with 2 cu. yd. concrete buckets, vibrators and other such equipment. This article gives a brief survey of the range of research now in progress at Jackson. Special attention is paid to the question of partial replacement of cement for the purpose of reducing to a safe minimum the amount of cement used in C.o.E. structures. Portland blast-furnace slag cement is also the object of extensive research. Field exposure tests are carried out at two special stations: Treat Island in Cobscook Bay near Eastport, Maine, and St. Augustine, Florida. Grouting, and especially the pumpability of sanded grouts, is also under investigation with the help of regular field mixing and pumping equipment. Other subjects under study are instruments for measuring pore pressures developed when a reservoir is flooded, the effectiveness of pozzolans as inhibitors to the expansion due to alkali-aggregate reaction, the creep characteristics of concrete, permeability of concrete to water, etc. (Thomas B. Kennedy, Chief Concrete Division Waterways Experiment Station, Vicksburg, Miss., *Engineering News-Record*, Vol. 159, No. 8, August 22, 1957, p. 34, 3 pp.)

Hydro-Electric Development in Italy

This is a highly interesting survey, by one of Italy's foremost experts, of the present state of electric power development in Italy, including considerations on some technical aspects and a review of recent developments. Following an outline of the general situation, the author discusses the co-ordination of hydro and thermal resources, and reviews the aspects and tendencies of Italian water-power technique, particular stress being laid on large dams. (Dr. Ing. Carlo Semenza, SADE, Hydraulic Construction Branch, *Power Engineer*, Vol. 7, No. 2, April 1957, p. 56, 7 pp., 4 ff.)

Analysing Water-Hammer Waves

A variant to the Bergeron graphical method of analysis is suggested. Instead of linking the successive states of pressure and flow in a pipe to the initial condition, the author deduces them one from the other by introducing one or more "differential disturbances" during each time interval. An account of the discussion which followed the reading of this paper is appended. (Professor A. Schlag, Liège University, *Mémoires and Travaux de la Société Hydrotechnique de France*, published as an issue of *La Houille Blanche*, Vol. XII, No. 4, May-June 1957, p. 296, 7 pp., 4 ff.)

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